GEOTECHNICAL INVESTIGATION AND SLOPE DESIGN FEASIBILITY STUDY FOR THE OTJIKOTO GOLD MINE IN NAMIBIA

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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, 2017

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.

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12 May 2017

ABSTRACT

This research report focuses on the geotechnical analysis and slope design for the construction of the proposed Otjikoto Gold Mine. The report focuses on the characterisation and analysis of the sub-surface conditions in the vicinity of the proposed gold mine to the final pit depths. Laboratory testing data together with geotechnical drillhole log data assist with the geotechnical investigation. The analysis of the sub-surface conditions is assessed using the Laubscher (1990) Mining Rock Mass Classification System. Results from the data analysis are used to derive values that are used for limit equilibrium analyses and results derived are then confirmed using numerical methods. Once the optimum slope angles are determined a consequence based rock fall risk analysis is undertaken. As the focus of the study was based on a classic rock mass classification system, strength and experience is added to the knowledge base regarding this system. This allows for a reference point to situations requiring similar solutions.

To my first GODS, Mum and Dad,

And to my significant other, Neeresha Pillay

For your eternal love, affection and support…

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- B2Gold, for allowing me use of the data obtained from their proposed mine site;
- My family, friends and colleagues, for their ongoing support and encouragement.
- Gratitude goes to my divine master, the one that makes everything possible, thank you my Lord.

Table of Contents

List of Tables

List of Figures

1 INTRODUCTION

In 2012, B2Gold Namibia requested a mining geotechnical study that incorporated all geotechnical drillhole data as well as data from a laboratory testing programme for the definitive feasibility study slope design. The Otjikoto gold project is an open pit mining project focused on the mining of the gold mineralisation that is hosted by a series of thin (<10 cm) sheeted veins, which lie fundamentally parallel to the northeast trending foliation of the schists and granofels (metamarls) of the basal Oberwasser Formation. Data for the study was made available from the geotechnical holes drilled specifically for the project. This research report addresses the geotechnical nature of the intact rock, the joint strengths as well as the rock mass strengths and an analysis of bench, inter ramp or stack stability as well as overall slope stability. The Otjikoto gold project is located approximately 300 kilometres north of Windhoek, Namibia's capital city between the towns Otjiwarongo and Otavi and is currently in the construction stage (Figure 1-1).

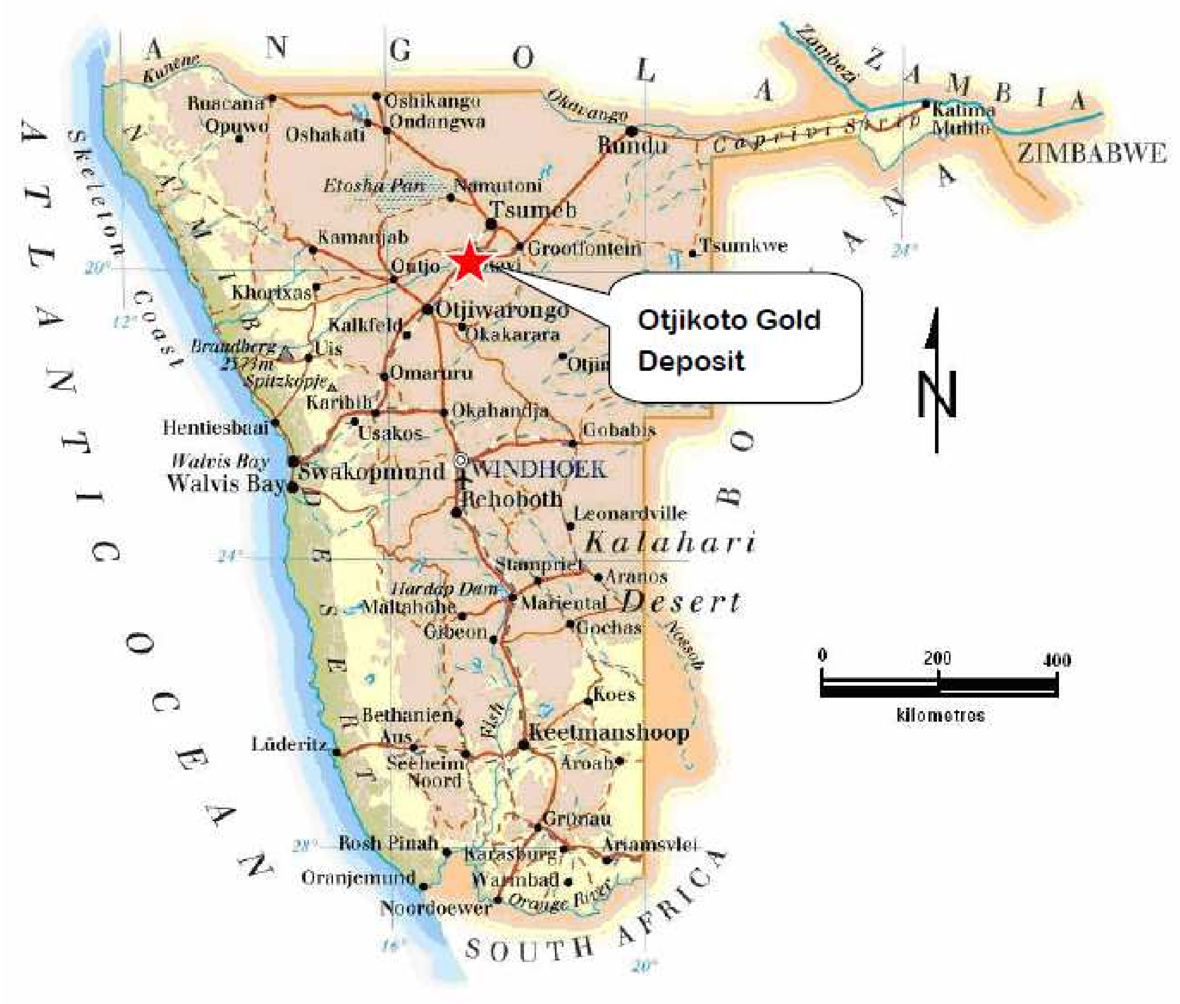


Figure 1-1: Otjikoto Project Location (SRK, 2010)

This research report has been prepared to the standard of, and is considered by the author to be, a technical assessment report under the Guidelines for Open Pit Slope Design published by the Commonwealth Scientific and Industrial Research Organisation (Read and Stacey, 2008). The design procedure follows the process recommended by the guidelines and is presented as Figure 1-2.

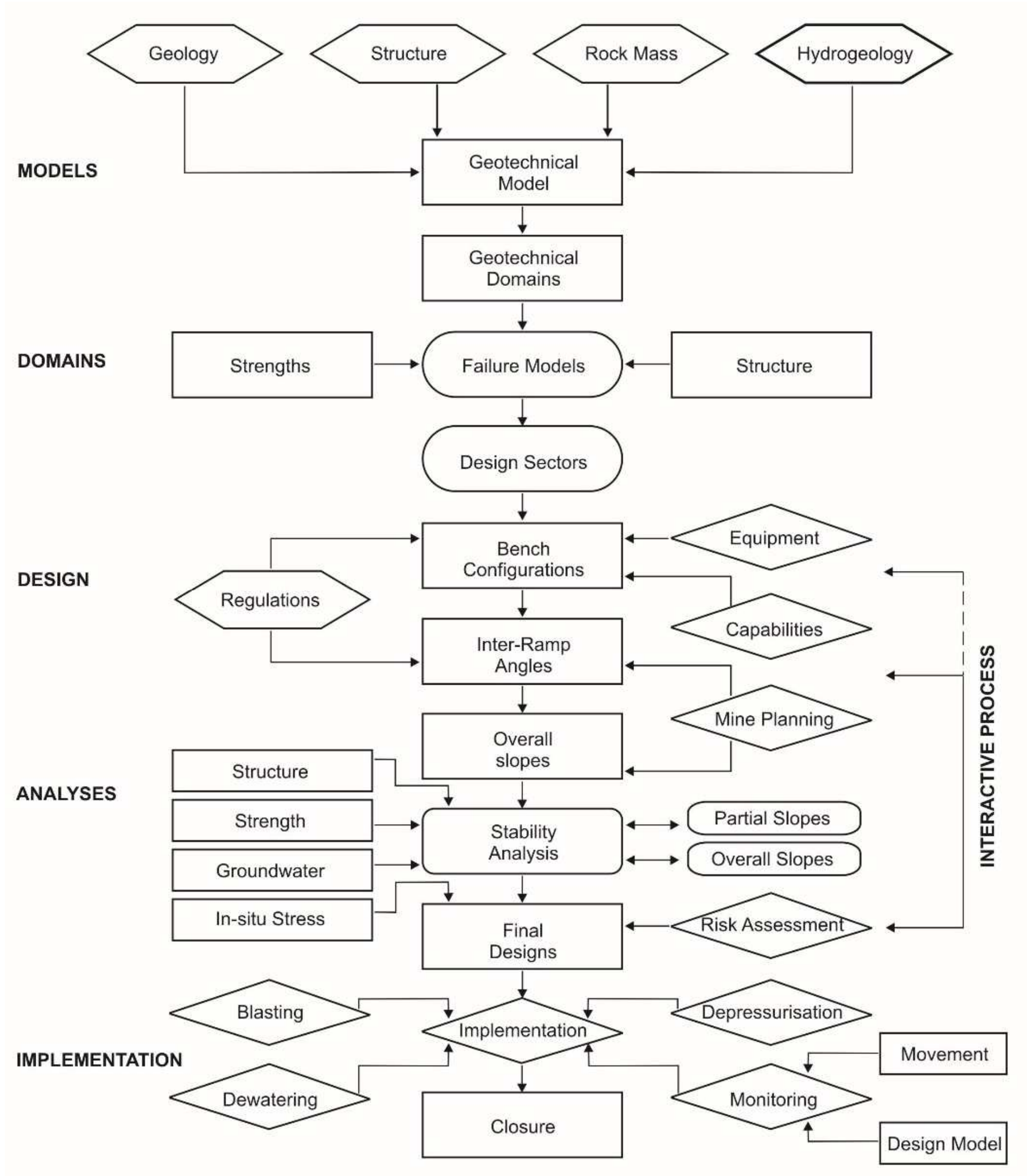


Figure 1-2: Slope Design Procedure (Brown and Booth, 2008)

The objective of this research project is to undertake a geotechnical investigation for the design and construction of the Otjikoto gold mine to a depth of 230 m, which is determined to be the final pit depth. The geotechnical investigation includes the classification and analysis of the quality of the sub-surface conditions observed in the vicinity of the Otjikoto gold mine area. The nature and quality of the sub-surface ground conditions were derived from data collected from the geotechnical drillhole logs as well as from laboratory testing results. The current geological models and the comprehensive databases were thereafter assimilated from the geotechnical drillhole logs and laboratory testing programme in order to determine the optimum pit slope angles, together with design recommendations and a risk assessment pertaining to rock fall for the proposed Otjikoto pit.

In order to investigate and classify the quality of the rock mass, the rock mass classification system used was Laubscher's (1990) Mining Rock Mass Rating Classification System. Laubscher's (1990) mining rock mass rating values provide a classification of the rock mass aimed to verify and support the design achieved.

This research topic was chosen because it allows the opportunity to provide B2Gold with in depth information and knowledge of their sub-surface conditions in the vicinity of the Otjikoto gold mine. As Laubscher's (1990) Mining Rock Mass Rating System (MRMR) approach is utilised in the study, strength and experience is added to the current knowledge base concerning the use of this system, thus allowing a reference point for solutions to similar situations. In addition to the rock mass classification system 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 depth of 230.00 m.
- To classify the rock mass in the vicinity of the Otjikoto gold mine footprint in terms of Laubscher's (1990) Mine Rock Mass Rating (MRMR) system.
- To identify any adverse geotechnical ground conditions that may impact the stability of the rock mass by conducting a limit equilibrium analysis and finite element analysis using the Rocscience software SLIDE and PHASE2 and undertaking a risk assessment using the software package ROCFALL.
- To provide the optimal design slope angle recommendations for the final pit slopes.
- To demonstrate the use and the adjustments that may be made to the widely used 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 presents an introduction to the research topic, a statement by the author as to why the research was carried out, and includes the research objectives.

Chapter 2 presents 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 the Otjikoto gold mine is presented. The geological setting, geotechnical drilling programme and structural evaluation results are discussed here.

Chapter 4 presents the rock mass classification approaches utilised with respect to Laubscher's MRMR and Hoek's quantification of the GSI system, the laboratory testing analysis is also presented here.

In Chapter 5 the design strength parameters are defined.

Chapter 6 includes the slope stability analysis of the final design slope angles and the rock fall analysis.

In Chapter 7, conclusions and recommendations derived by the author are offered and discussed.

All research report references are listed as Chapter 8.

2 LITERATURE REVIEW

Previously presenting a concise introduction to the research topic in Chapter one, a thorough literature review of the various rock mass classification systems with specific interest in the philosophy, application and advancement of these systems is presented in this Chapter.

The need for a suitable classification system in the field of rock mechanics has long been recognised and, in fact, numerous proposals have been made (Bieniawski, 1973). Bieniawski (1973) went on to elaborate that the classification system should be:

- Based on the rocks inherent properties,
- Useful in practical design, and
- Use terminology that is widely acceptable.

A classification technique must be straightforward, so that it may form part of normal geological investigations (Laubscher, 1990). Engineering classifications of rock masses are recognised today as an essential adjunct for assessing rock mass conditions for engineering purposes (Bieniawski, 1976). Rock mass classification methodologies are commonly used today as a useful means of estimating rock mass stability, support requirements in underground openings, rock mass deformability and rock mass strength. Rock mass classification systems thus form the backbone of the empirical design approach and are widely used today (Bieniawski, 1989). Rock mass classification systems have proven to be a powerful tool in rock engineering applications and have been highly beneficial since their introduction into the industry.

The accuracy of the geomechanics classification system utilised depends on the sampling of the area of investigation (Laubscher, 1990). With that being stated, it is imperative to note that if the systems are not utilised with caution, inaccurate results are likely to be produced. Sound engineering judgement and experience are therefore critical when employing these systems to ensure success.

A review of geotechnical literature indicates that numerous formal classification systems have been put forward and developed since the year 1946. Bieniawski (1973) identified some difficulties associated with the development of a reasonable rock mass classification system and that classification systems:

- Tended to be based entirely on the rock characteristics.
- Were too general to facilitate an objective evaluation of rock quality.
- Were not practical.
- Did not provide quantitative information on the properties of rock masses.
- Emphasised the characteristics of discontinuities, but disregarded the properties of intact rock material.
- That did not include information on rock mass properties and which, therefore, could only be applied to one type of rock structure.

These issues apart, nine classifications are described below, which illustrate the evolution of rock mass classification systems over time. Each of the systems represents a leap forward in the mission to develop a satisfactory rock mass classification system.

2.1 Rock Mass Classification

2.1.1 Terzaghi's Rock Load Height Classification

In 1946 Terzaghi published the earliest reference on the use of rock mass classification for the design of support systems in tunnels. This classification system was descriptive in nature, based on the characteristics that dominated the rock mass behaviour where gravitational force constitutes the dominant driver. The system was made up of seven rock mass descriptors which are listed below:

- Intact rock
- Stratified rock
- Moderately jointed rock
- Blocky and Seamy rock
- Crushed but Chemically intact rock
- Squeezing rock
- Swelling rock

The approach was widely utilised in the United States of America for over 25 years and employs the use of steel supports, but it is not suitable to modern day tunnels that apply shotcrete and install rock bolts as a form of support (Bieniawski, 1973). Additionally, as the interpretation of the rock condition is based on the user's opinion, it is considered qualitative or semi quantitative. Cecil (1970) established that Terzaghi's classification was too general to allow an objective evaluation of rock quality and that it provided no quantitative information on the rock mass properties. For these reasons this system is rarely used today.

2.1.2 Lauffer's Stand Up Time Classification System

Lauffer's (1958) classification system has its roots in the earlier research on tunnel geology conducted by Stini (1950). Lauffer (1958) proposed that the stand-up time for an unsupported span relates to the quality of the rock mass in which the span is excavated. An unsupported active span is the width of the tunnel or the distance from the face to the support if this is less than the tunnel width. The stand-up time is defined as the period of time that the tunnel will stand unsupported after excavation (Lauffer, 1958). The original classification set out by Lauffer has been revised on numerous occasions by other Austrian Engineers, most notably Pacher, von Rabcewicz and Golser (1974) and now forms part of the general tunnelling approach known as the New Austrian Tunnelling Method.

The core significance of Lauffer's classification system is that it shows how an increase in a tunnel span leads to a drastic reduction in the stand-up time. For example, a pilot tunnel with a small span may be successfully completed in a fair rock mass, but a larger span opening in the same rock mass may not be stable without the addition of immediate support (Figure 2-1) (Lauffer, 1958). Figure 2-1 defines the rock mass classes by letter with "A" representing very good rock and "G" representing very poor rock.

The disadvantage of the classification system is that the two parameters are difficult to establish and thus much is demanded of practical experience. Nonetheless, the concept introduced the stand-up time and the span as the two most relevant factors for the determination of the type and the amount of tunnel support, and this has paved the way forward for the more recent rock mass classification systems (Bieniawski, 1990).

The method that supersedes Lauffer's system, namely 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 methods include the use of smaller headings and benching or the use of multiple drifts, resulting in a reinforced ring inside which the bulk of the tunnel may be excavated. These techniques are applicable in both soft rock and in excessively broken rock, though considerable care should be taken when applying these techniques in hard rock where different failure mechanisms occur (Hoek et al. 1995)

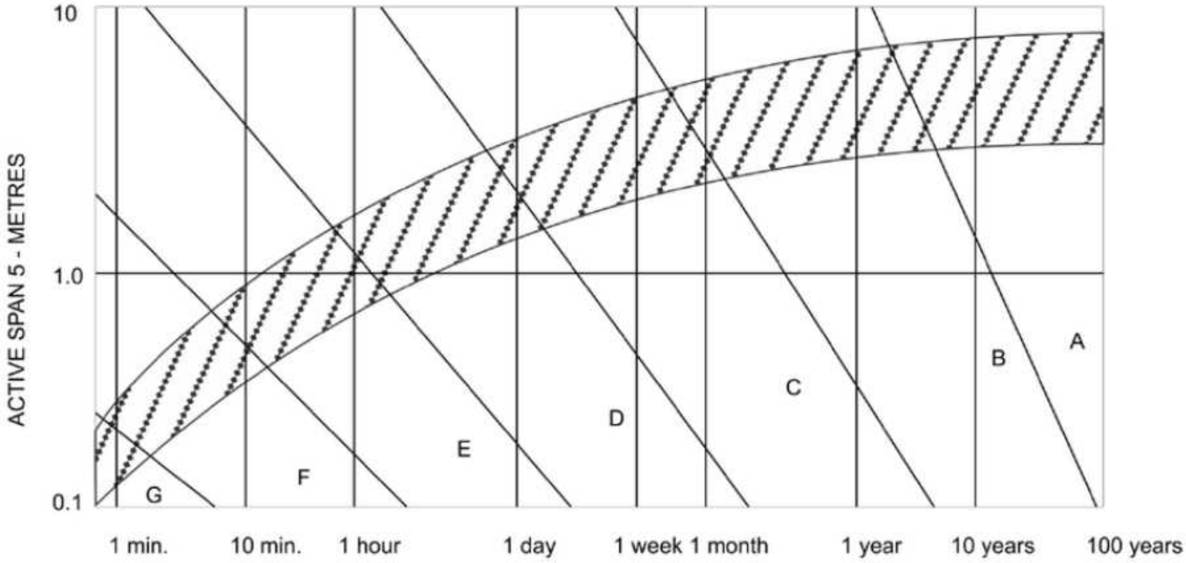


Figure 2-1: Lauffer's relationship between the active span a stand-up time for different rock mass classes (After Lauffer, 1958)

2.1.3 Rock Quality Designation Index

The Rock Quality Designation (RQD), was initially proposed by Deere (1964) as an index method of assessing rock quality quantitatively in civil engineering applications (Lucian and Wangwe, 2013). The RQD now features as a commonly used index for the description of the rock mass fractured state that has been rapidly implemented in mining, geotechnical engineering and engineering geological applications (Lucian and Wangwe, 2013).

A brief background

Deere (1964) developed the RQD concept for assistance in the design of tunnels and large caverns in granitic rock at a Nevada test site in the United States of America. This concept was extended the following year to the design of highway tunnels in gneiss, schist and massive quartzite rock in North Carolina (Deere and Deere, 1988). The University of Illinois pioneered the development of the RQD concept to feature in a broader array of rock engineering problems. A chapter in Rock Mechanics in Engineering Practice (Deere, 1968) introduced RQD to an international audience, thus leading to the acceptance of the concept and its use in many countries (Deere and Deere, 1988).

Methodology

The Rock Quality Designation is a modified core recovery percentage, in which all the pieces of solid core over 10 cm (100mm) in the total length are summed up and divided by the length of the core run (Deere and Deere, 1989). The minimum requirements for applying the RQD index method include:

- The diameter of the core should not be less than 54.7 mm in diameter (NX Size)
- The use of double-tube core barrel drilling

The RQD is most often used as a standard geotechnical core logging parameter and provides a rapid and inexpensive index value of rock quality in highly weathered, fractured, soft, sheared and jointed rock masses (Edelbro, 2003). In its simplest form, RQD is a measurement of the percentage of the good rock. Since only intact core is considered in the measurement of RQD, weathering is accounted for in an indirect manner (EM 1110-1-2908, 1994). The correct procedure for measuring RQD is presented in Figure 2-2.

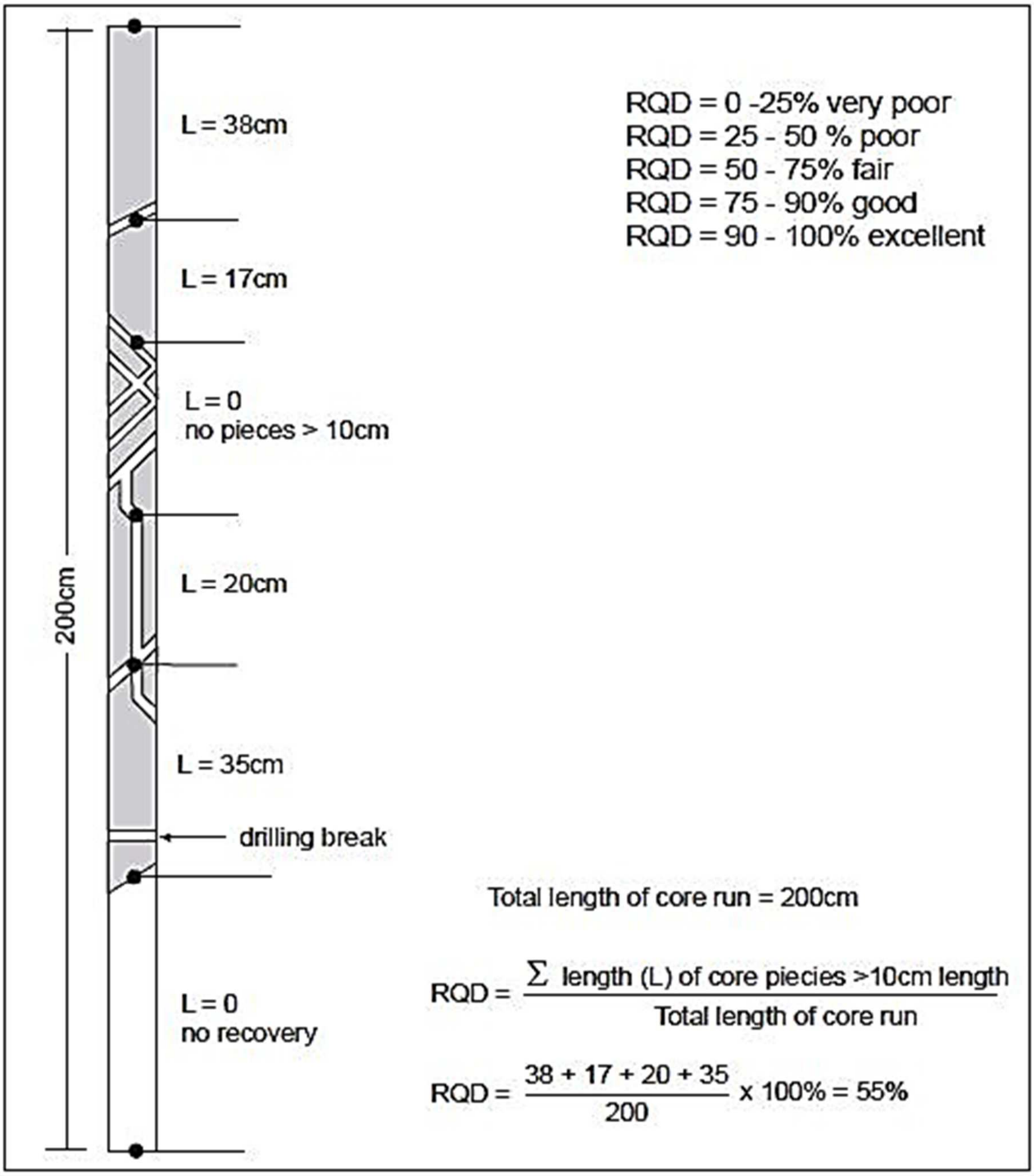


Figure 2-2: Measurement and Calculation of RQD (after Deere, 1989)

In order to effectively make use of the RQD system, the following practices should be applied:

- Length measurements of the core should be along the centreline (Figure 2-3). Where mechanical breaks of the core occur due to handling or otherwise, the core must be fitted together and counted as a continuous piece.
- Preferably 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. In instances where the core is laminated, soft, comprise of 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 sub-divided 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 are recommended (Deere and Deere, 1989).

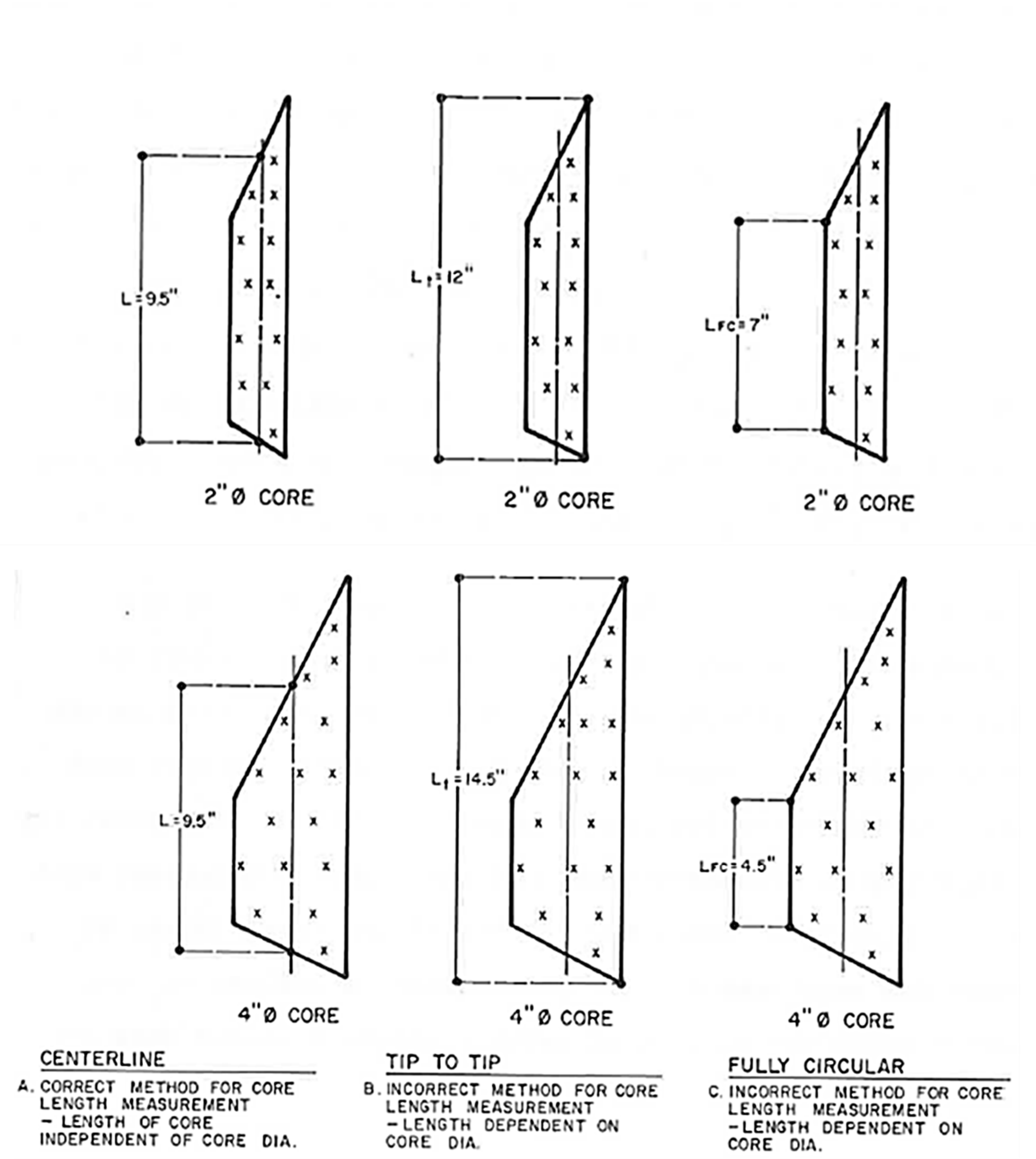


Figure 2-3: RQD measurement (Deere and Deere, 1989)

The relationship between the RQD index and the rock mass quality that was proposed by Deere (1964) is presented in Table 2-1.

Table 2-1: The Relationship between RQD and Rock Mass Quality

Rock Quality Designation (%)	Rock Mass Quality
< 25	Very Poor
25 < 50	Poor
50 < 75	Fair
75 < 90	Good
90 < 100	Excellent

On its own, RQD has become internationally recognised as an indicator of rock mass conditions and is most commonly known as an input parameter for both the Rock Mass Rating (RMR, MRMR) and Q-System Classification Systems.

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 drillhole. This is clearly demonstrated in Figure 2-4. Figure 2-5 shows three extreme cases, where RQD is either equal to 0 or 100 depending on the orientation of the drillhole, 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 critically for the “red flagging” of zones with poor quality rock. This is likely the most valuable quality of this approach. While the RQD system should not be used in isolation for rock mass classification, RQD does provide an excellent input parameter for other popular rock mass classification systems (e.g. Bieniawski’s (1989) Rock Mass Rating system and Barton’s (2002) Q system). The RQD system is therefore likely to be used for many years to come, and should thus be applied diligently with the recommended techniques for logging in mind.

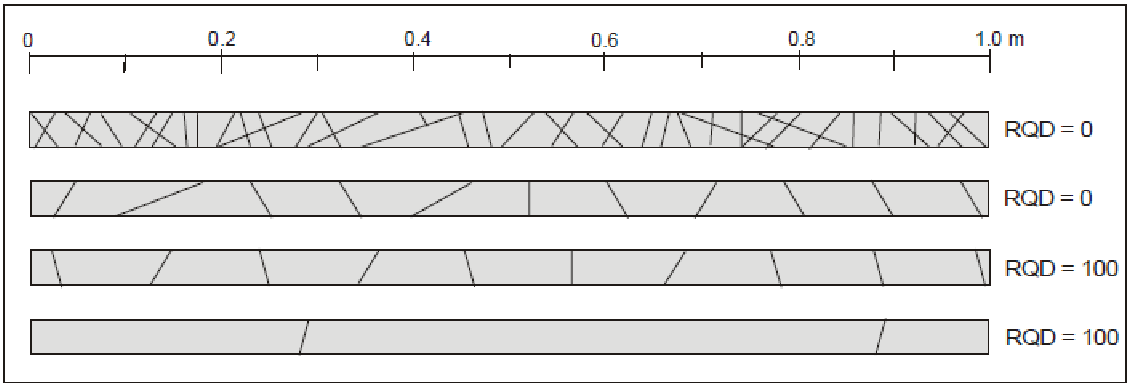


Figure 2-4: Limitations of the conventional method of determining RQD (Palmstrom, 2005)

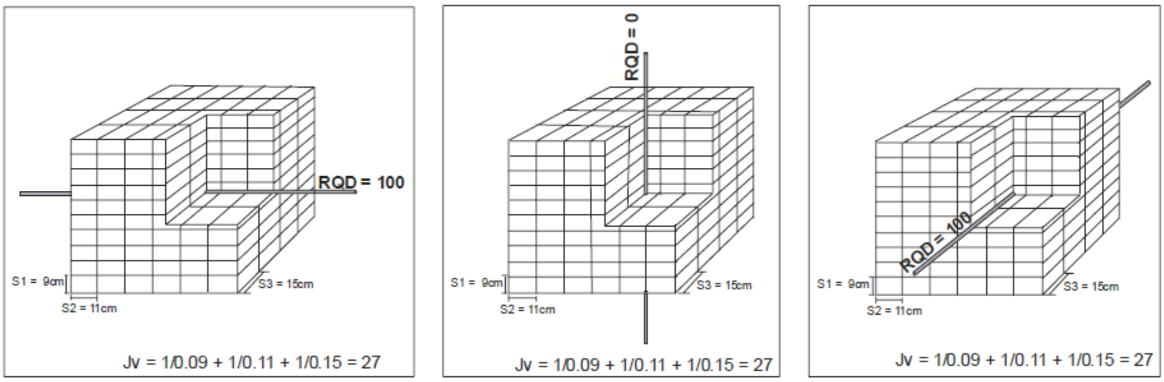


Figure 2-5: Drillhole orientation bias (Palmstrom, 2005)

Palmstrom's correlation between Volumetric Joint Count and RQD

The joints that intersect 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 a function of the number of joints; joint spacing 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 therefore used 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.
- GSI (Geological Strength Index) system, where block size is expressed in varying degrees compared with the weathering of joints to produce a GSI value.

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

Types of Block Size Measurements

Measurements of joints are predominantly made on 2-dimensional surfaces and along 1-dimensional drillhole lines or scanlines, even though they are three dimensional in nature (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, adits, shafts 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 2-2 illustrates the main methods used to measure block size.

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

Rock Surfaces	Drill Cores	Refraction Seismic
Block Volume (Vb)	Rock Quality Designation (RQD)	P-Wave 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).

Volumetric Joint Count

A volumetric joint count (Jv) method was developed by Palmstrom (1974). The Jv is a 3-dimensional measurement for the density of joints. It is therefore 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:

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

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 is applied where random joints exist:

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

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

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

Where observed, random joints should be accounted for, as these joints represent part of the number of discontinuities present (Palmstrom, 2005). Furthermore, they a may play an influential role in the overall behaviour of the rock mass.

The classification of Jv is presented in Table 2-3.

Table 2-3: Classification of Jv (modified after Palmstrom, 2005)

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

Rock Quality Designation using Jv

As noted earlier, RQD was initially developed by Deere (1964) to offer a quantitative estimate of rock mass quality from drill core. Since RQD is a one dimensional measurement, based solely on core pieces 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:

$$\boldsymbol{RQD = 115 - 3.3Jv}$$

Where: *RQD* = 0 for Jv > 35

RQD = 100 for Jv < 4.5

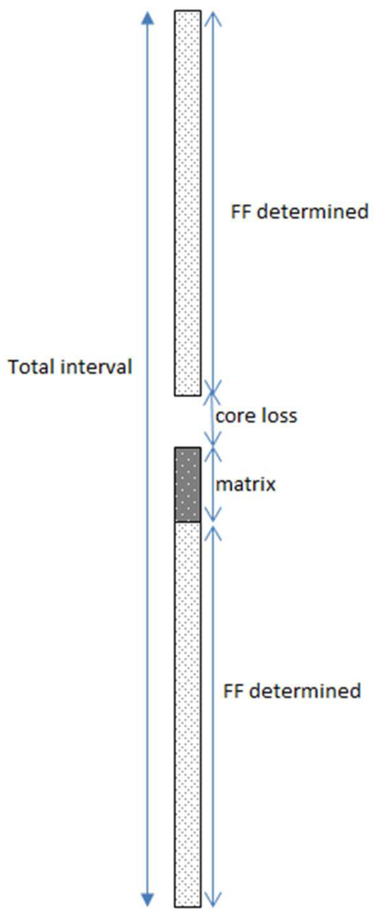
In recent times, Palmstrom (2005) extended his research of the RDQ correlation with Jv with the inclusion of computer generated blocks of varying shapes and sizes. Concluded from the research, Palmstrom (2005) suggested the following modification to his equation:

$$RQD = 110 - 2.5Jv$$

The new correlation between RQD and Jv was found to provide somewhat better results than the initial RQD = 115 -3.3Jv relationship.

As Palmstrom's approach to determine RQD does not take core loss and the length of matrix into account (Figure 2-6), a correction factor can be applied into the Palmstrom equation which reduces the RQD in intervals where core loss and matrix is present:

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



Core Recovery = Interval length – core loss

Figure 2-6: Geotechnical interval depicting core loss and presence of matrix

2.1.4 Rock Classification for Rock Mechanics Purposes (Patching and Coates, 1968)

This system saw the modification of the Coates (1964) and Coates and Parsons (1966) classification of rock (Edelbro, 2003). The system considers two stages (after Patching and Coates, 1968):

1. The actual rock substance
2. The rock mass.

The classification contained five categories, of which two related to the rock mass and three related to the rock substance, this aided in the subdivision of rocks into different classes. These categories are presented as Table 2-4.

Table 2-4: Rock Classification Categories

Rock Substance	**1. Geological Name of the Rock**
	2. Uniaxial Compressive Strength of the Rock Substance
	(a) Very low (<27.5 MPa)
	(b) Low (27.5 - 55 MPa)
	(c) Medium (55 -110 MPa)
	(d) High (110 - 220 MPa)
	(e) Very High (>220 MPa)
	3. Pre-failure Deformation of Rock Substance
	(a) Elastic
	(b) Yielding
Rock Mass	**4. Gross Homogeneity of Formation**
	(a) Massive
	(b) Layered
	5. Continuity of the Rock Substance in the Formation
	(a) Solid (joint spacing >1.8m)
	(b) Blocky (joint spacing 0.9 - 1.8m)
	(c) Slabby (joint spacing 0.08 - 0.9m)
	(d) Broken (joint spacing <0.08m)

2.1.5 Rock Structure Rating (Wickham et al. 1972)

The system defined a quantitative method for describing the rock mass quality and furthermore recommended appropriate support on the basis of their Rock Structure Rating (RSR) result (Wickham et al. 1972).

The system was developed using historical case histories involving the development of relatively small tunnels that were supported by steel sets. The RSR system is also credited with the accolade of being the first to make reference to shotcrete as a support (Hoek, 1995). The RSR system is an important system to study, as it demonstrates the ideology of the logic involved in the development of a rating concept to arrive at a numerical value for RSR at the end of the process.

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 parameters defined above are classified as follows:

The geology parameter (A) accounts for the general appraisal of geological features such as:

- The origin of the rock (igneous, sedimentary, metamorphic)

- The hardness of the rock (decomposed, soft, medium, hard) and
- The rock fabric (massive, slightly/moderately/intensely faulted/folded)

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

- Joint spacing,
- Strike and dip of joints (orientation), and
- Direction of tunnel advance (Figure 2-7).

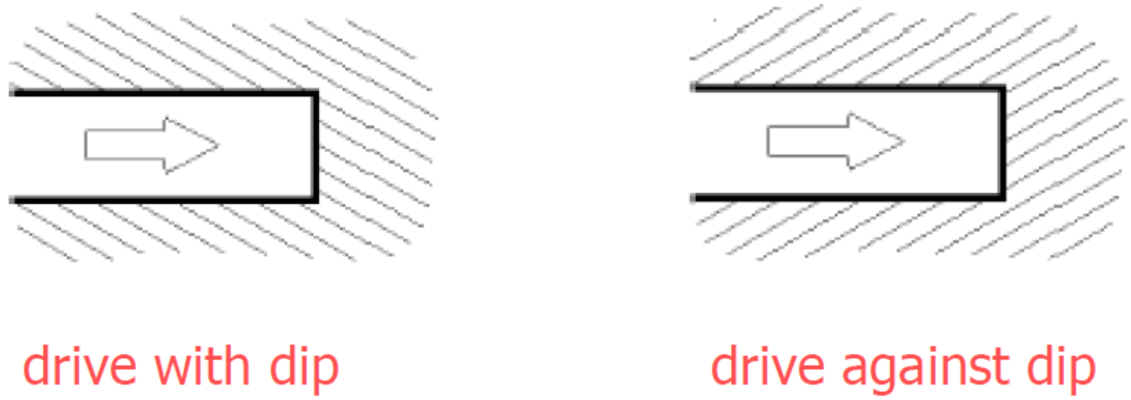


Figure 2-7: Illustration depicting drive with and against dip with associated tunnel advance (After Hoek, 2007)

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

- The quality of the rock mass as derived from the combination of parameters A and B,
- The joint condition (good, fair, poor), and
- The amount of water inflow into a tunnel (in gallons per minute per 1000 feet of tunnel).

The tables used to appraise the parameters developed by Wickham et al. (1972), to calculate the resultant RSR value are presented as Table 2-5 to Table 2-7. The maximum RSR value obtainable is 100. RSR classification made use of Imperial units which are valid for the tables below.

Table 2-5: RSR Parameter A (after Wickham 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 2-6: RSR Parameter B (after Wickham 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[1]					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 2-7: RSR Parameter C (after Wickham et al. 1972)

Anticipated water inflow gpm/1000 ft of tunnel	Sum of Parameters A+B					
	13 – 44			45 – 75		
	Joint Condition[2]					
	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

[1]Dip: flat: 0-20°; dipping: 20-50°; vertical: 50-90°

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

2.1.6 Bieniawski’s Rock Mass Rating System

The Geomechanics classification system, known as the Rock Mass Rating (RMR) classification system, was introduced in 1973 by Bieniawski. Bieniawski (1973) saw the need to design a system to meet both practical applications and ensure effective communication between the engineer and the geologist or the designer and contractor. Bieniawski (1973) stated that any rock mass classification system shoud satisfy five rudimentary requirements, namely:

- A classification system should be based on inherent rock properties that are quantifiable and easily determined in the field.
- The system should be useful in practical design.
- The terminology used in the system should be broadly accepted.
- The classification system should be general enough so that the same rock could possess the same classifcation, irrespective of how it was being utilised.
- The tests and observations required for the purpose of classification should be rapid, simple and relevant.

Bieniawski was of the view that the systems that were proposed up to 1973 had not fully satisfied these five rudimentary requirements. He found that the systems had two primary limitations that needed to be addressed. These were:

- Most classification systems were based wholly on the characteristics of the rock mass, and therefore were impractical.
- The systems that were practical did not include information on the rock mass properties, and could therefore only be applied to a single type of rock structure.

Bieniawski (1973) developed the RMR classification system by combining the best features from the classification systems available at the time and went on to declare that a rock mass classification should:

- Divide a rock mass into groups of similar behaviour;
- Provide a good basis for understanding the characteristics of the rock mass;
- Facilitate the planning and the design of structures in rock by acquiring quantitative data required for the solution of real engineering problems; and
- Provide a common basis for effective communication among all persons concerned with a geomechanics problem.

It was further suggested that these aims should be fulfilled by ensuring that the chosen 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).

Bieniawski 1973

The classifcation system proposed by Bieniawski (1973) includes the following parameters:

- **The Rock Quality Designation (RQD)**

  RQD was chosen as it provides an indication of the in-situ quality of the rock mass. Additionally, a relationship exists between RQD index value and fracture frequency recorded from geotechnically logged drill cores.

- **Degree of weathering**

  Weathering and alteration play significant roles in the behaviour of a rock mass. Bieniawski (1973) considered five classes of weathering which are employed in the system. These weathering classes are defined in the table below (Table 2-8):

Table 2-8: Degree of weathering (After Bieniawski, 1973)

Unweathered	There are no visible signs of weathering, the rock is fresh, and the crystals are bright, possible slight staining associated with some discontinuity surfaces may exist.
Slightly Weathered:	Associated with penetrative weathering that developed on open discontinuity surfaces, there is slight weathering of the rock material, discolouration of the discontinuities is possible up to a maximum of 10mm from the discontinuity surface.
Moderately Weathered:	The greater part of the rock mass is slightly discoloured, the rock material is not friable (exceptions are poorly cemented sedimentary rocks), the discontinuities are stained and/or contain altered material.
Highly Weathered:	The rock material is friable with weathering extending throughout the rock mass, the rock lacks lustre, all material besides quartz is discoloured, rock can be excavated by a geologist’s pick.
Completely Weathered:	The rock is entirely decomposed, discoloured and friable, only fragments of the original rock fabric and structure are preserved, material has a soil like appearance.

- **Uniaxial Compressive Strength (UCS) of Intact Rock**

  Five classes as proposed by Deere (1966) are considered for the system. Bieniawski (1973) was of the opinion that this proposed strength classification is both realistic and convienient for its proposed use. A modified version of Deere's classification is presented as Table 2-9. The classification was modified for the purpose of conforming to rounded values of the International System of units (SI).

Table 2-9: Strength Classification of Intact Rock (Bieniawski, 1973)

Description	Uniaxial Compressive Strength (MPa)
Very Low Strength	1 - 25
Low Strength	25 - 50
Medium Strength	50 - 100
High Strength	100 - 200
Very High Strength	> 200

The system allows for the intact rock strength to be determined either by conducting uniaxial compression strength tests in a laboratory or point load strength tests (an index test) in the field. The strength classification of intact rock has come under scrutiny from various parties who claim that no subdivisions are given below 25 MPa. Bieniawski (1973) purposefully considered this conservative approach as strength values under 25 MPa do not play a role in the overall mobility of a rock mass.

- **Spacing of Discontinuities**

  The presence of joints reduces the strength of a rock mass. The closer the joint spacing, the lower the rock mass strength and vice versa. The Rock Mass Rating System (1973) considers five classes of joint spacing after Deere (1968). This modified classification is presented as Table 2-10.

Table 2-10: 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 stability of rock slopes is significantly influenced by the inclination of the discontinuity surface in which the rock is excavated. These discontinuities can be in the form of bedding planes, foliation, joint, fracture, cleavage, schistosity, fracture, fissure, crack or fault plane. Bieniawski (1973) preferred practical solutions and the assessment for the RMR system involves evaluating whether the dip and strike conditions of the rock mass are favourable or unfavourable for the proposed excavation.

- **Joint Separation**

  This parameter is not often considered in rock mass classification systems. However, Bieniawski (1973) included this as it is considered important for the quantitative assessment of a rock mass. Hutchinson and Diederichs (1996) also confirmed that it is a practical condition for the quantitative description of a rock mass, due to closely spaced jointing forming smaller block sizes which increase the potential for internal shifting and rotation of the rock mass during deformation thereby reducing stability.

- **Continuity of Joints**

  The continuity of joints will affect the degree to which rock material and joints affect a rock mass. Persistant joints have a higher probablity to combine with other structures to form large free blocks of rock (Hutchinson and Diederichs, 1996).

- **Groundwater inflow**

  Groundwater can have a weakening effect on a rock mass, usually through the erosion and weakening of joint surfaces and/or infillings (Hutchinson and Diederichs, 1996). In simpler terms, where groundwater exists, the strength of the rock mass is reduced. Bieniawski (1973), inculded this parameter into the RMR classification system and states that rock mass classification systems should account for the influence of groundwater.

In Bieniawski's (1973) research it became evident that all the parameters described contributed to the behaviour of a rock mass, however, not all parameters were of equivalent importance. To account for this Bieniawski (1973) allocated a weighted numerical rating based on the parameter's relative importance. In essence, higher ratings were associated with better geotechnical conditions and vice versa. Bieniawski (1973) simplified the classification system by sub-dividing the rock mass into five classes, which he thought sufficient to provide acceptable, clear differences between the different rock material qualities (Dyke, 2006). The Geomechanics classification of joint rock masses (After Bieniawski, 1973) is presented as Table 2-11 and the summary of the relative importance of Bieniawski's individual parameters is presented as Table 2-12.

Table 2-11: 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 mm	< 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 10m 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 Favourable
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 2-12: Individual Ratings for classification Parameters (after Bieniawski, 1973)

Parameter		Class				
		1	**2**	**3**	**4**	**5**
RQD		16	14	12	7	3
Weathering		9	7	5	3	1
Intact Rock Strength		10	5	2	1	0
Joint Spacing		30	25	20	10	5
Joint Separation		5	5	4	3	1
Joint Continuity		5	5	3	0	0
Ground water		10	10	8	5	2
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

Application of the RMR System requires that the rock mass be sub-divided into geotechnical zones. Geotechnical zones can be described as areas or zones of a rock mass that are confined by major structural features, significant changes in discontinuity spacing, changes in lithology or changes in characteristics. The rock mass is characterised according to the parameters laid out in Table 2-12, and the sum of the individual parameters produces the total RMR rating value, which is then used to establish the class of rock.

Changes to the RMR System

The RMR System proposed by Bieniawski in 1973 can be considered somewhat of a prototype to the systems that we have today. The system has been refined over the years (with the addition of more case histories) with significant changes made in 1974, 1975, 1976 and 1989 respectively. It must be noted that when quoting an RMR value, it is imperative that the version of the system used to derive the value is also stated.

The changes made to the RMR System are based on a better understanding of the importance of the respective parameters, and with the addition of newer case studies, modifications to the system were implemented. The changes also brought about the application of the RMR System into the realm of open pit mining, with it being used for the preliminary design of rock slopes, as well as for the approximation of in-situ modulus of deformation and rock mass strength.

The major changes made to the system over the years are presented as Table 2-13.

Table 2-13: Revisions made to RMR system

Year of Revision	Revision made
1974	Joint condition parameter added
	Dip Orientation and Strike parameter added
	Removal of strike and dip orientation for tunnels, and removal of the weathering, joint separation and continuity parameters
	RQD parameter weighting increased from 16 to 20
1975	Original joint condition parameter rating increased from 15 to 30
	Rock Strength Parameter rating increased from 10 to 15
	The strike and dip orientation parameter for tunnels was reintroduced, but changed from 3 - 15 to 0 – 12
	The dip orientation and strike parameter added in 1974 was removed
1976	Joint condition parameter rating revised to 25
	Rock Mass classes introduced in 1973 were sub-divided into intervals of 20
1989	Joint condition parameter rating increased back to 30
	Weighting of the discontinuity spacing parameter was decreased to 20
	Groundwater weighting parameter increased to 15
	Discontinuities condition descriptions were further quantified
	The assessment of sub-horizontal joints was altered from “unfavourable" to "fair", this accounts for the effect on stability of tunnel backs.

Strengths and Limits of the System

- The principal strength of the RMR System is the ease with which it can be applied. The system is simple to use and the data can be easily acquired from mapping or drillhole data.
- The RMR System is adaptable and can be applied to coal mining, hard rock mining, slope stability, tunnelling and foundation stability.
- As part of the development of the Hoek-Brown failure criterion, the RMR value was linked to the original equation (Hoek and Brown, 1980).
- A shortcoming of the system is that the system has been found to be unreliable in very poor rock masses (Singh and Goel, 1999).
- The output is found to be too conservative for the mining industry which tends to lead to over design in terms of recommendations regarding support.

The present RMR System (1989) by Bieniawski is presented as Table 2-14.

Table 2-14: 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	-

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		

2.1.7 Laubscher's Mining Rock Mass Rating Classification System (MRMR) (1990)

Building on from what Bieniawski developed in 1974, Laubscher went on to develop the completely independent Mining Rock Mass Rating (MRMR) System in 1976. Thereafter this was superseded by the Mining Rock Mass Rating Classification System in 1990, also developed by Laubscher. This system has been used extensively in the mining industry for over 25 years.

The system also makes use of a ratings system, with geological parameters weighted according to their relative importance. The maximum possible resultant rating is out of 100. The rating values range between 0 and 100 and are sub-divided into five rock mass classes made up of ratings of 20 per class. The classes range from very poor to very good, which is in line with the concept introduced by Bieniawski (1989). The difference in the Laubscher (1990) class is that the rock mass can be further sub-divided into a division A and B.

The Mining Rock Mass Rating System employs the following parameters:

- Intact rock strength (IRS)

  This parameter refers to the Uniaxial Compressive Strength (UCS) of the intact rock between fractures. Rating associated with the IRS is presented as Table 2-15.

Table 2-15: Intact Rock Strength Ratings Table (After Laubscher, 1990)

Parameter	**Range of Values**										
UCS (MPa)	185	184-165	184-165	164-145	144-125	124-105	104-85	84-65	44-25	24-5	4-0
Rating	20	18	16	14	12	10	8	6	4	2	0

- Rock Quality Designation (RQD)

  The percentage RQD is considered. This is calculated by:

$$RQD\ (\%) = Total\ Length\ of\ Core > 100mm\ \div Length\ of\ Run\ \times 100$$

  Rating associated with the RQD is presented as Table 2-16.

Table 2-16: Rock Quality Designation Ratings Table (After Laubscher, 1990)

Parameter	**Range of Values**								
RQD	100-97	96-84	83-71	70-56	55-44	43-31	30-17	16-4	3-0
Rating	15	14	12	10	8	6	4	2	0

- Fracture or Joint spacing

  This parameter refers to the measurement of all partings and discontinuities (except cemented fractures/joints). This parameter considers a maximum of three joint sets. In the event of there being more than three joint sets, the three closest-spaced joint sets are utilised (Laubscher, 1990). A method to evaluate the joint spacing rating is presented as Figure 2-8.

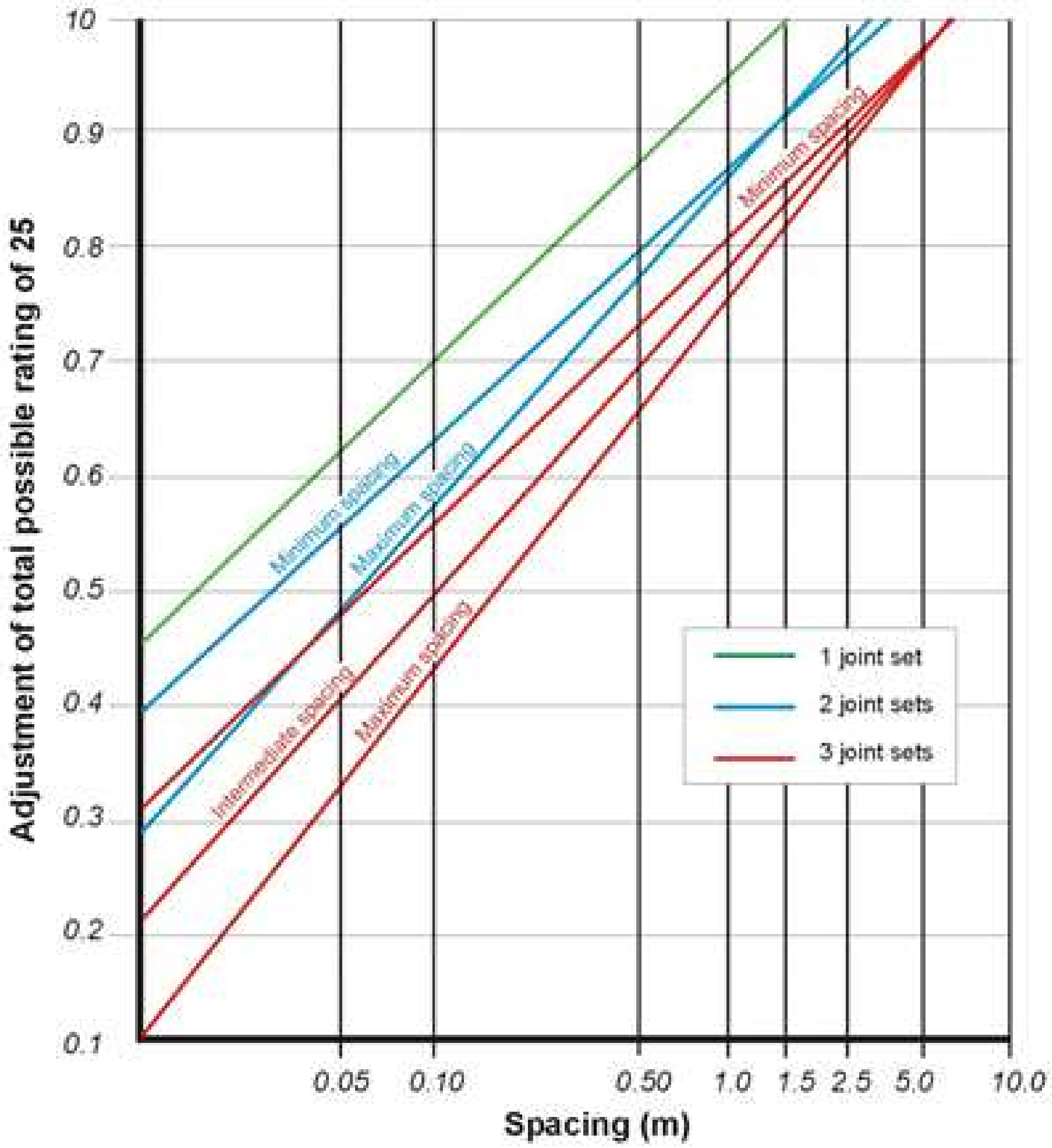


Figure 2-8: Graph for Joint Spacing Rating (After Laubscher, 1990)

- Joint Condition and Groundwater

  The joint condition parameter in essence is the evaluation of the frictional properties of the joints based on surface properties, expression, alteration zones, infill material and water (Laubscher, 1990). Laubscher (1990), developed a system to assess the joints, this system is reflected as Table 2-17.

  The ratings associated with the joint conditions can have a maximum possible value of 40. The assessment considers four adjustment criteria and also makes provision for groundwater by downgrading the ratings based on the water pressure. Large scale irregularities are considered in section A, section B accounts for the small scale irregularities. The joint wall alteration is considered in section C and the joint infill material is classified according to section D. The system accounts for differing joint conditions for each set by utilising a weighted average rating value (Laubscher, 1990).

Table 2-17: Groundwater and Joint Condition Rating (Laubscher, 1990)

Parameter	Description		Dry Condition	Wet Conditions: Moist	Wet Conditions: Moderate pressure 25-125 1/mm	Wet Conditions: Severe Pressure >125 1/min
A Joint Expression (large scale irregularities)	Wavy	Multi-Directional	100	100	100	95
	Wavy	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	

The process of determining the MRMR value can now be facilitated by taking the RMR value achieved from the above process and applying adjustments based on the mining environment effects. The factors considered are weathering, mining induced stresses, blasting damage and orientation of joints. The adjustments are based on several field observations and as such are empirical (Laubscher, 1990).

- Weathering

Laubscher (1990) stated that weathering disturbs three of the Rock Mass Rating parameters, namely IRS, RQD and joint condition. The weathering adjustments defined by Laubscher (1990) made provision for a period of six months to four years. These adjustments are presented as Table 2-18.

Table 2-18: Adjustments for weathering (After Laubscher, 1990)

Description	Potential Weathering and Percentage Adjustments				
	6 Months	1 Year	2 Years	3 Years	4+ Years
Fresh	100	100	100	100	100
Slightly Weathered	88	90	92	94	96
Moderately Weathered	82	84	86	88	90
Highly Weathered	70	72	74	76	78
Completely Weathered	54	56	58	60	62
Residual Soil	30	32	34	36	38

- Mining Induced Stresses

Mining activities cause the redistribution of regional stress fields, this in turn results in mining-induced stresses. These resultant stresses play a role in the stability of an excavation based on the shape and depth of the excavation. Laubscher (1990) accounted for these stresses by allowing adjustments based on poor and good confinement conditions. Poor confinement was attributed an adjustment of 60% and good confining conditions were attributed 120%, the latter being the result of improved stability conditions.

- Blasting Damage

The effect of blasting in the mining environment needs to be considered. Laubscher (1990) stated that blasting creates fresh fractures, loosens the rock mass and also causes movement along the existing joints. The MRMR system accounts for blasting by considering four blasting techniques. These techniques are defined as:

- Boring
- Smooth wall blasting
- Good conventional blasting
- Poor blasting

The adjustments laid out by Laubscher (1990) are presented as Table 2-19.

Table 2-19: Adjustments for blasting (After Laubscher, 1990)

Technique	Adjustment (%)
Boring	100
Smooth Wall Blasting	97
Good Conventional Blasting	94
Poor Blasting	80

- Orientation of Joints

A rock mass may be made up of intact rock only, but is more likely to be formed from an array of intact rock blocks with limits formed by discontinuities (Hack, 1996). Joint orientation affects the stability of an excavation. Laubscher (1990) defined adjustments based on the orientations of the joints with respect to the vertical axis of the block. The percentage adjustments pertinent to joint orientations are presented as Table 2-20.

Table 2-20: Joint orientation adjustment table (After Laubscher, 1990)

Number of Joints Defining the Block	No of Faces Inclined Away from the Vertical				
	70%	75%	80%	85%	90%
3	3		2		
4	4	3		2	
5	5	4	3	2	1
6	6	5	4	3	2,1

The four adjustments defined above are applied cumulatively as multipliers to the initial RMR rating value achieved. When this process has been followed, the ultimate result will be the MRMR. When the system is correctly applied, the results are highly suitable to be utilised.

2.1.8 The Norwegian Geotechnical Institute's Q-System (Barton et al. 1974)

The Norwegian Geotechnical Institute (NGI) Q-system was based on an analysis of approximately 200 tunnel case studies which showed a significant correlation between the type and amount of permanent support and the quality of the rock mass with regard to tunnel stability (Barton et al., 1974). The original database used for the development of the classification system is summarised as Table 2-21.

Table 2-21: 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

The system makes use of six parameters in determining the quality of the rock mass. The rock mass quality is determined by using the following equation:

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

Where: *RQD* is the rock quality designation (in percentage)

Jn is the joint set number

Jr is the joint roughness number

Ja is the joint alteration number

Jw is the joint water reduction factor

SRF is the stress reduction factor

The three quotients defined in the above equation can be described as:

- RQD/Jn: Barton et al. (1974) described this quotient as representing the rock mass structure, and results in a crude estimate of block size.
- Jr/Ja: is described as representing the frictional features and roughness of the joint walls, this quotient also gives an indication of the inter-block shear strength.
- Jw/SRF: this factor is a complicated quotient that relates two stress parameters. This gives a crude indication of the active stress conditions. The SRF is considered to represent the total stress parameter, and is a measure of loosening load in the case of excavation through shear zones and clay rich rocks, rock stress in a competent rock and squeezing loads in incompetent plastic rock masses. The Jw is a water pressure parameter; this parameter has a negative effect on shear strength of joints through the reduction of effective normal stress (Barton et al. 1974).

The resultant Q value derived from the calculation varies on a logarithmic scale that ranges from 0.001 to 1000, with the rock mass quality divided into nine classes. The summary of the nine classes are presented as Table 2-22.

Table 2-22: Q-System Classification System (After Barton et al. 2002)

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 noted that both the Q and the RMR Systems consider three primary rock mass properties (Dyke, 2006):

- Intact Rock Strength
- Frictional properties of Discontinuities
- Geometry of the intact blocks of rock as demarcated by the discontinuities

Although similarities exist between the two systems based on the consideration of the three primary properties, cognisance must be taken of the fact that these two systems are not directly related. However, a number of authors have derived correlations between the Q system and the RMR system. Most prominent correlations were established by Bieniawski in 1976 and Barton in 1995, with the former being the more popular equation. The equation derived by Bieniawski (1976) linking the two systems is defined as:

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

This Bieniawski (1976) comparison was based on 117 case histories which involved 68 Scandinavian, 21 United States and 28 South African operations (Goel et al. 1996)

The Barton (1995) correlation equation is defined as:

$$RMR = 15 \log Q + 50$$

The summary of correlations representing the relationship between RMR and Q by a number of authors who have also derived similar correlations for specific applications is presented as Table 2-23.

Table 2-23: Q and RMR System Correlations (after Milne et al. 1998)

Correlation Equation	Source	Application
RMR = 13.5 log Q + 43	New Zealand	Tunnels
RMR = 12.5 log Q + 55.2	Spain	Tunnels
RMR = 5 ln Q + 60.8	South Africa	Tunnels
RMR = 43.89 – 9.9 ln Q	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

Advancement of the Q-System

Since the inception of the original Q- System, several updates and improvements have been made. The system is now based on 1050 case histories with Grimstad and Barton (1993) publishing a technical paper titled “Updating of the Q-System for NMT”. This paper made provision for adjustments related to narrow weakness zones, updates the SRF values table, proposes the application of new rock support methods. Barton published a technical paper in 2002 titled “Some New Q-value Correlations to Assist in Site Characterisation and Tunnel Design”. A brief summary of the advancements made to the Q- System is presented as Table 2-24. This paper introduced a significant number of changes to the respective Q-System parameters. The Barton (2002), amended Q-System parameters are presented from Table 2-25 to Table 2-30. It is important for the user to specify the version of the system used when quoting a result for Q.

Table 2-24: Advancements 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.

Table 2-25: Rock Quality Designation (Barton, 2002)

Rock quality designation		RQD (%)
A	Very poor	0–25
B	Poor	25–50
C	Fair	50–75
D	Good	75–90
E	Excellent	90–100

Notes: (i) Where RQD is reported or measured as ⩽10 (including 0), a nominal value of 10 is used to evaluate *Q*. (ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate.

Table 2-26: 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 2-27: 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 2-28: 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 2-29: 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 ⩽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 ⩽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 $SIGMA_{cm} \approx 5\gamma Q_c^{1/3}$ (MPa) where γ is the rock density in t/m^3, and $Q_c = Q \times \sigma_c/100$, Barton [29].

Table 2-30: 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.

Support Recommendations

This empirical approach of determining the rock mass quality helps in estimating support requirements. The determination of support based on the Q-System requires the determination of the equivalent dimension of the excavation. This is accomplished by dividing the span/height of the excavation by the Excavation Support Ratio (ESR). The ESR is based on the structure's intended use (Barton et al. 1974). The ESR can simply be defined by, the smaller the value the more sensitive or critical the excavation (e.g. nuclear power station).

Support recommendations can be made based on plotting the equivalent dimension relative to the Q value on Figure 2-9 and by doing so, an estimation of the reinforcement categories can be made. The categories defined in Figure 2-9 consider rock bolts, shotcrete liners and bolt patterns

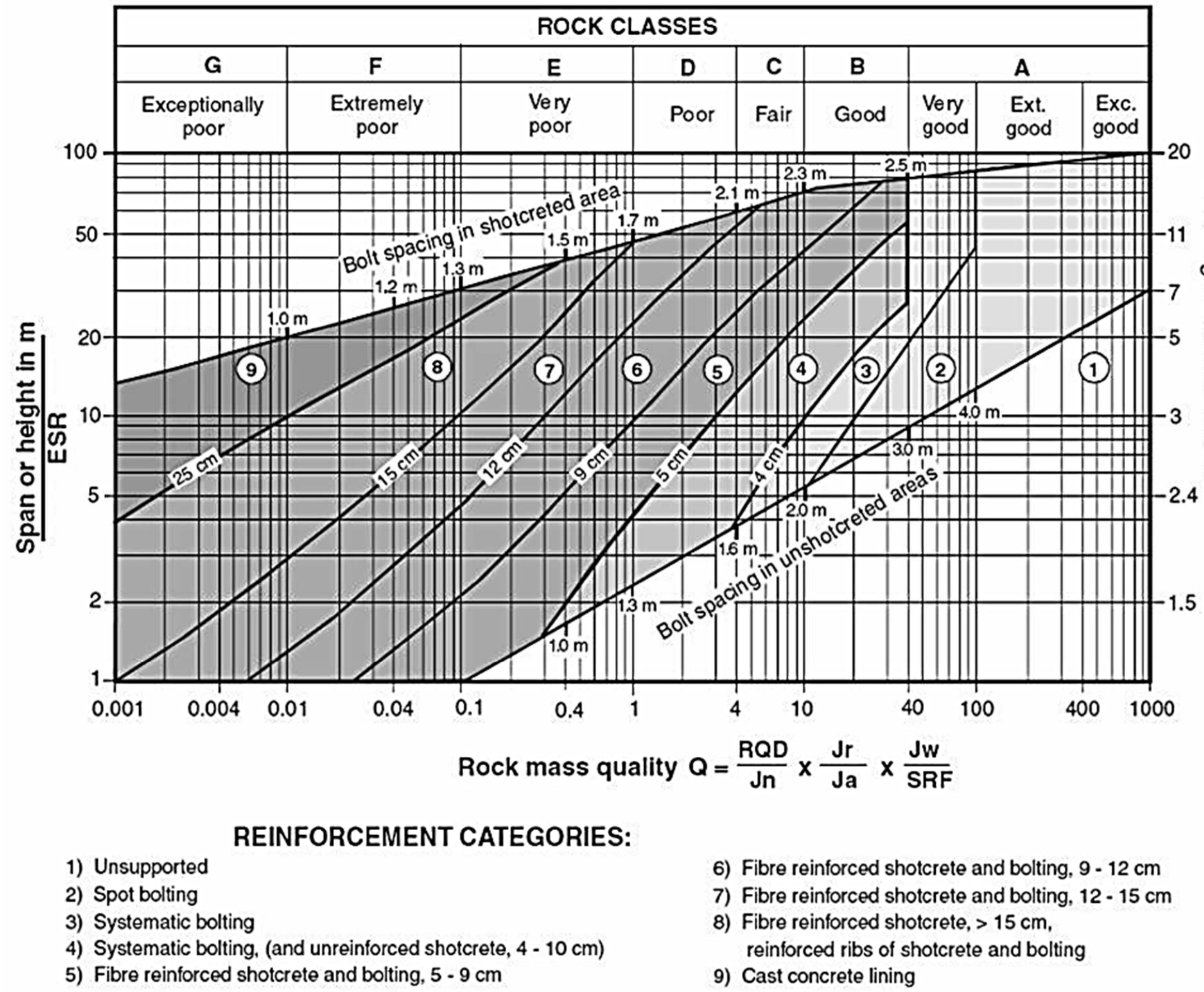


Figure 2-9: Support recommendations based on Q (After Grimstad and Barton, 1993)

Palmstrom and Broch (2006), deliberated on the uses and misuses of the Q-System. Some of their remarks echoed that the system should be used with caution. Palmstrom and Broch (2006), were of the opinion that the user of the Q-System needs to have the essential understanding and experience to evaluate the parameters required for the system to work effectively. Other authors, such as Milne et al. (1998) found that inexperienced users experienced difficulty with the J_n parameter. This is particularly relevant in the widely jointed rock masses where the overestimation of the number of joint sets poses an underestimation of the Q-value.

The Q-System tends to work best in the area emphasized in Figure 2-10.

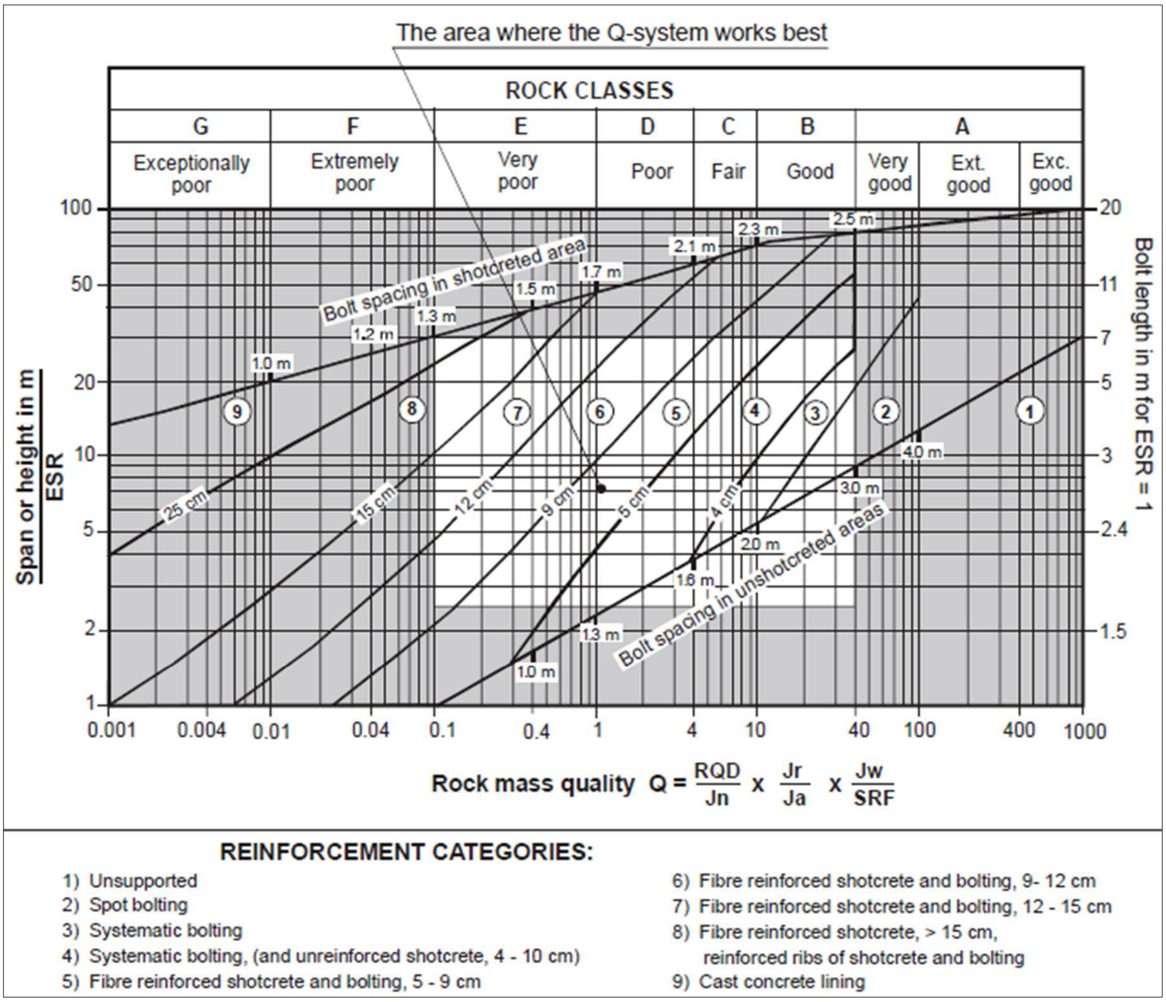


Figure 2-10: Limitations of the Q-System Support Chart (Palmstrom and Broch, 2006)

Palmstrom and Broch (2006) found that the use of the Q-System can be of immense benefit during the feasibility and the initial design phases of a project, as little detailed information is available at this stage. At its most simplistic state, the Q-System can be used as a checklist in ensuring that all important information is collected and considered (Palmstrom and Broch, 2006).

2.1.9 Geological Strength Index (GSI)

The Geological Strength Index (GSI) has become as synonymous as RMR, MRMR and Q to the practitioners that make use of rock mass classification systems. Hoek et al., (2005) felt that the widely used RMR and Q-Systems are heavily reliant upon the RQD result that is based on Deere's (1964) work. A consensus that RQD is basically zero in weak rock masses prompted the development of a system that was not reliant on RQD.

Originally the GSI was developed as Hoek and Brown recognized that the rock mass criterion they have developed would have no applied value unless it could be linked to geological observations that could be made simply and rapidly by an engineering geologist or geologist in the field (Hoek and Marinos, 2000). The GSI was primarily meant to help engineering staff that were assigned to collect data, as these persons are usually less comfortable with qualitative descriptions. The GSI chart is presented as Figure 2-11. It must be borne in mind that the GSI was never developed to replace the RMR or Q-Systems, as the GSI system's only function is for the estimation of rock mass properties

(Hoek et al. 2005). The GSI can simply be said to be an assessment of the structure, lithology, and discontinuity surface condition of a rock mass that is estimated from observations made of exposed rock mass outcrops, in surface excavations and from drillhole cores (Hoek et al. 2005).

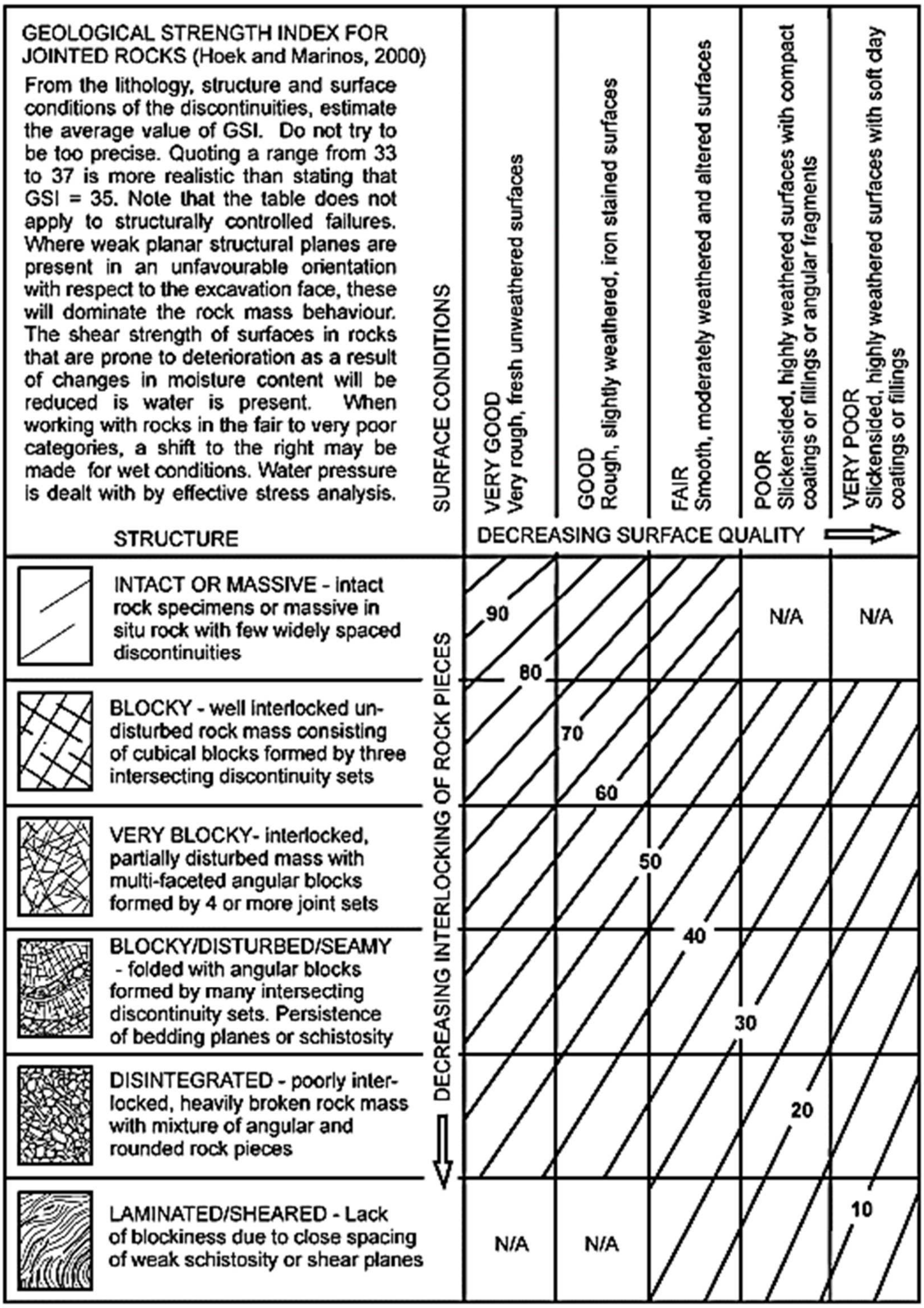


Figure 2-11: General chart for GSI estimates from geological observations chart (Hoek and Marinos, 2000)

The chart comprises a description of a range of rock mass structures, also supplemented with an illustration of the structure being portrayed on the vertical axis, with the horizontal axis describing a range of the possible joint surface conditions.

Hoek et al. (2013), proposed a data flow chart for the process of using the GSI/Hoek-Brown method for the benefit of estimating parameters that are needed for numerical analyses regarding surface/underground excavations in rock. This flow chart describes a quantitative and qualitative approach to undertaking the GSI characterisation. This process is illustrated in Figure 2-12.

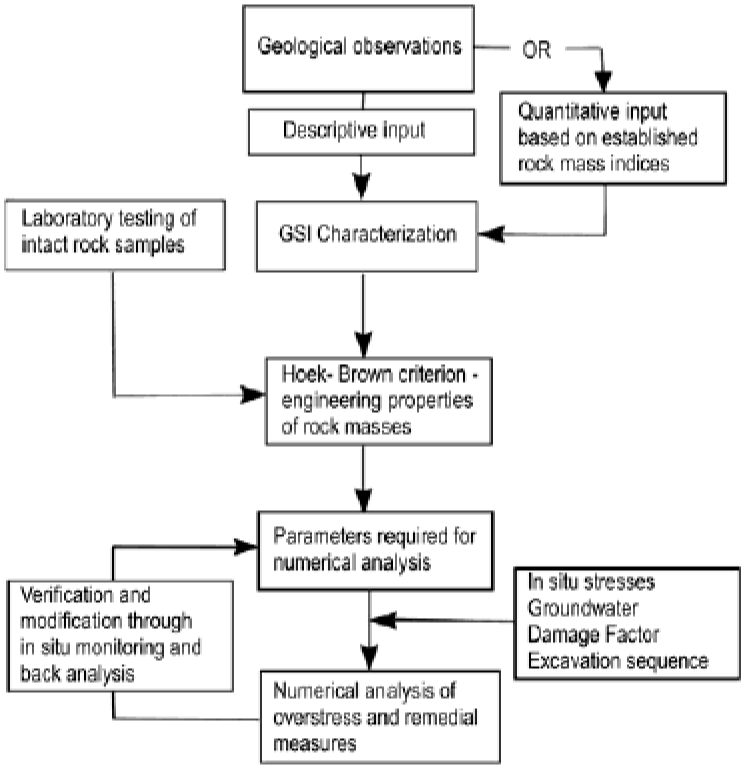


Figure 2-12: Data Process for using Hoek-Brown System (Hoek et al. 2013)

Previous work done by Hoek and Marinos (2000) saw the addition of two scales to the original GSI chart. 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). This edited chart is shown as Figure 2-13.

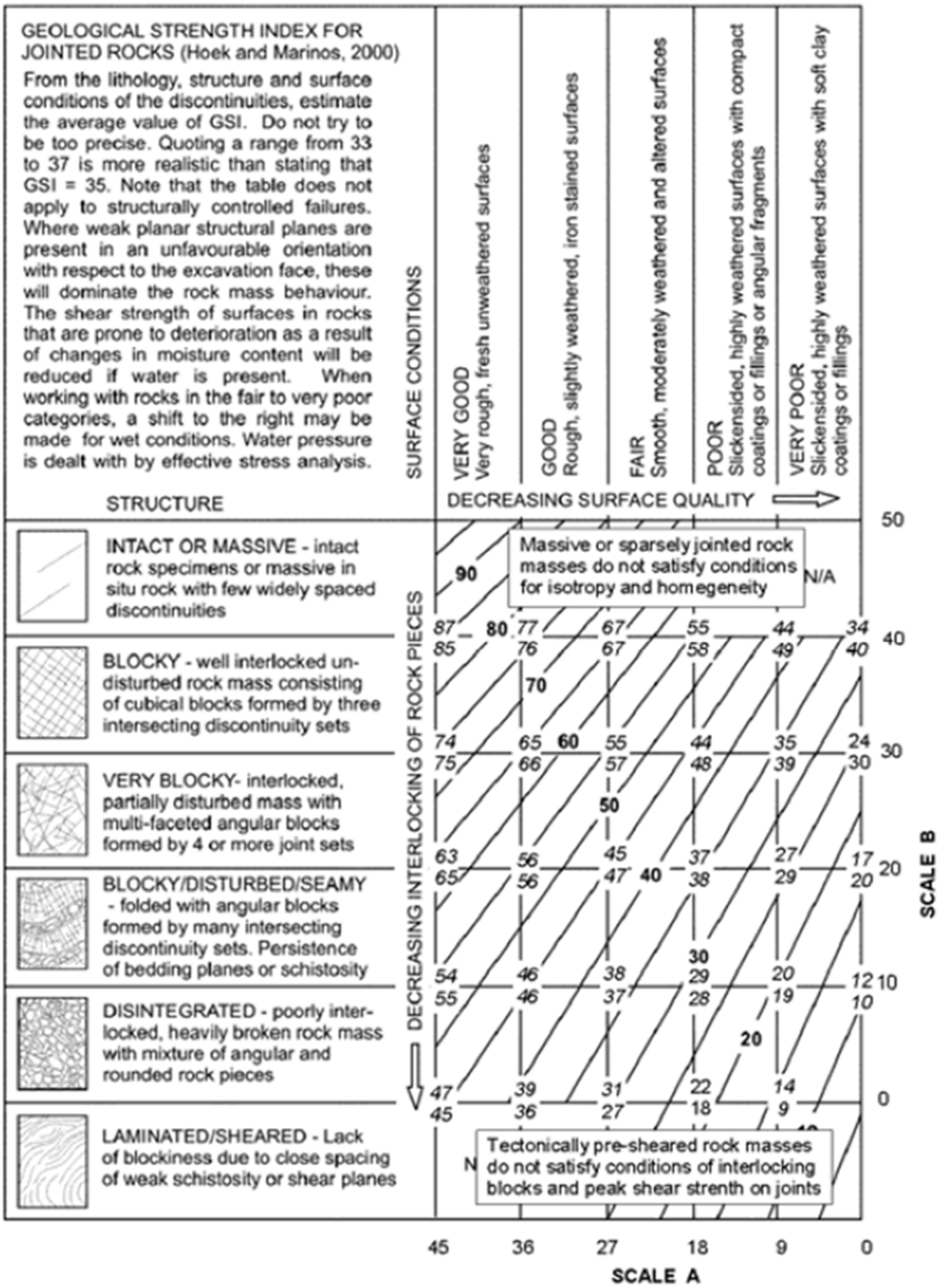


Figure 2-13: Original GSI chart with additional Scale A and Scale B (Hoek and Marinos, 2000)

The chart with the addition of the scales made it possible to quantify GSI by means of these linear scales representing the condition of discontinuity surfaces and the interlocking of the rock blocks that are bounded by these aforementioned discontinuities (Hoek et al. 2013).

Hoek et al. (2013) proceeded to make a minor adjustment to the original GSI chart by making the lines parallel and equally spaced. This was done to eliminate any errors as the original lines were hand drawn and not ideal, and the improved chart is presented in Figure 2-14.

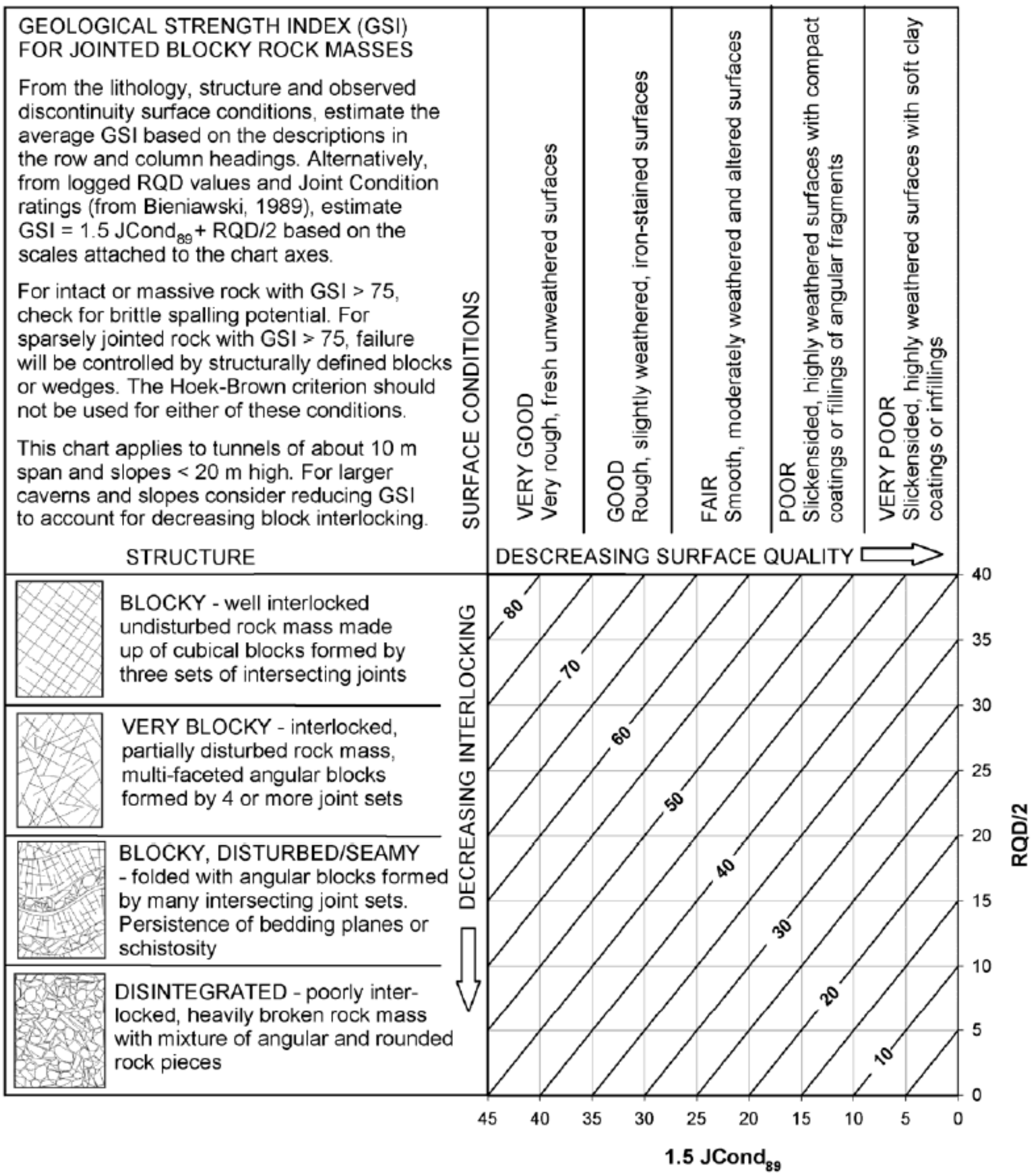


Figure 2-14: Updated GSI chart (Hoek et al. 2013)

The addition of the scales and the correction to the GSI lines do not affect the original function of the chart. This new improved chart satisfies both the descriptive and quantitative users (Hoek et al. 2013).

The improved GSI chart aids in providing a quantitative approach to the rock mass classification system. The following conditions and limitations apply:

- For an intact massive or very sparsely jointed rock mass, the GSI chart should not be used. This is because there are insufficient pre-existing joints to satisfy the conditions of homogeneity and isotropy. In order to avoid misunderstanding, the upper row of the improved chart has been removed (Hoek et al. 2013).
- The lower row of the original chart has also been removed. This is due to the row representing sheared, transported or heavily altered materials, which do not satisfy the conditions of homogeneity and isotropy (Hoek et al. 2013).

- There is an assumption that the rock mass is considered homogeneous and isotropic. Thus there should be several discontinuity sets that are sufficiently closely spaced, relative to the size of the structure under consideration as illustrated in Figure 2-15.

- It must be noted that the GSI system does not directly take into account the effects of water and stresses in the rock mass.

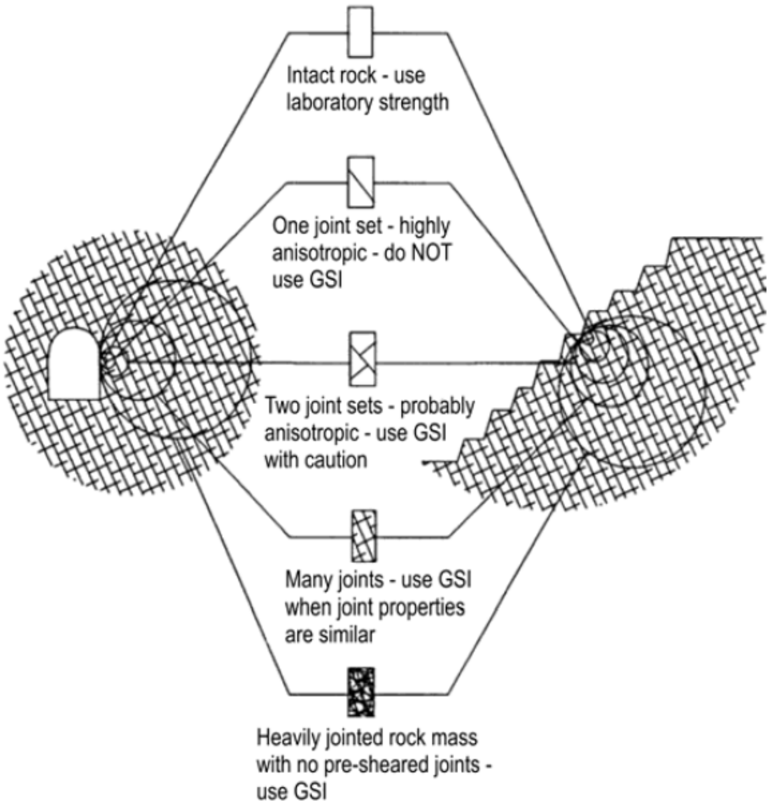


Figure 2-15: Limitations of GSI depending on scale factors (Hoek et al. 2013)

The estimation of GSI in terms of Joint Condition and RQD

With the addition of the two scales to the improved GSI chart, it became evident that the joint condition rating as defined by Bieniawski (1989) and the RQD as defined by Deere (1963) would be strong candidates that would relate to the scales (Hoek et al. 2013). The $JCond_{89}$ rating agrees well with the surface conditions defined in the text boxes of the horizontal-axis of the GSI chart. This rating parameter has been in use for many years and is found it to be both simple and reliable for application in the field. The RQD rating has been in use for over 50 years and are also found to be extremely reliable. These two ratings are therefore found to be ideal for use as the A and B scales for the quantification of GSI (Hoek et al. 2013).

Figure 2-14 demonstrates how scale A is defined by 1.5 $JCond_{89}$ while the B scale is defined as RQD/2. The value of GSI is given by the sum of these scales which results in the relationship:

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

Alternative joint scale or condition scale interpretation where values of $JCond_{89}$ are not available in data from field mapping for the surface quality axis have been investigated. One of these is a relationship between $JCond_{89}$ and the $JCond_{76}$ versions of this parameter, used in older data sets, which can be used as a replacement of $JCond_{89}$. Regression analyses found the relationship to be $JCond_{89} = 1.3\ JCond_{76}$ (Hoek et al. 2013). The substitution of the values present:

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

The second alternative is the quotient Jr/Ja from Barton et al. (1974) that gives a relationship to $JCond_{89}$ which provides an acceptable approximation for engineering applications. Hoek et al. (2013) stated the relationship is found to be $JCond_{89} = 35\ Jr/Ja/(1 + Jr/Ja)$. The substitution of the values present:

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

With minor modifications to the original GSI chart, it has been established that two simple linear scales, $JCond_{89}$ and RQD, can be used to characterise the discontinuity surface conditions and the blockiness of the rock mass. These ratings are well recognised in the field of engineering geology, are known to be simple to determine and are likely to give the highest degree of consistency between different geologists working on a single project. It is also found that this agreement is acceptable for the characterisation of jointed rock masses in order to attain properties for input to numerical models (Hoek et al. 2013).

2.2 Risk Method

The probability of occurrence of an event combined with the potential loss or consequence associated with that event can be defined as risk (Wilson and Crouch, 1987). This statement can be can be defined by:

$$RISK = P\ (event) \times Consequence\ of\ the\ event$$

The consequences associated with a slope failure were defined by Lilly (2000) as being the clean-up costs, the slope rehabilitation, repair of haul road, damage to equipment, personnel and infrastructure, and the sterilization of ore. The consequences defined can be grouped into two broad classes, the economic impact and the safety impact (Steffen, 1997). By adopting a risk approach to the design of a slope in the open pit mining environment, owners are effectively allowed to define their risk criteria, taking account of the specific consequences of potential failures, and the benefits of steeper slopes at higher risks, and therefore can commission the designers accordingly (Steffen et al. 2008).

The traditional design approach as defined by Steffen et al. (2008) is as follows:

- Collection of all geotechnical data that are required for the design to the appropriate confidence level based on the level of study/application
- Design the slope to a Probability of failure (POF) or Factor of Safety (FOS) criterion commonly used by geotechnical engineers.
- Provide the resultant slope angles to the mine planning team for their designs and economic derivations

Steffen et al. (2008) defined the risk/consequence design process as:

- Determining the risk criteria for each consequence at the inception
- Institute the best practice management tools for the slope's performance
- Determine the required POF for the design of the slope
- Design the slope to the necessary reliability at the required level of design
- Collection of geotechnical data suitable for the next required level of design confidence.

The methods available for developing a risk - consequence process are many, however, shared steps exist between them. The Australian Geomechanics Society (2000) described the guidelines as:

- Identifying the event that will generate the hazard

- Assess the probability of occurrence of these events
- Assess the impact of the hazard
- Combine the impact and probability to produce an assessment of risk
- Compare this calculated risk with benchmark criteria to produce a valuation of risk
- Use this valuation of risk as a tool to make a decision.

Figure 2-16 explains the risk approach to the open pit slope design process. This illustration (Figure 2-16) also incorporates the risk consequence philosophy.

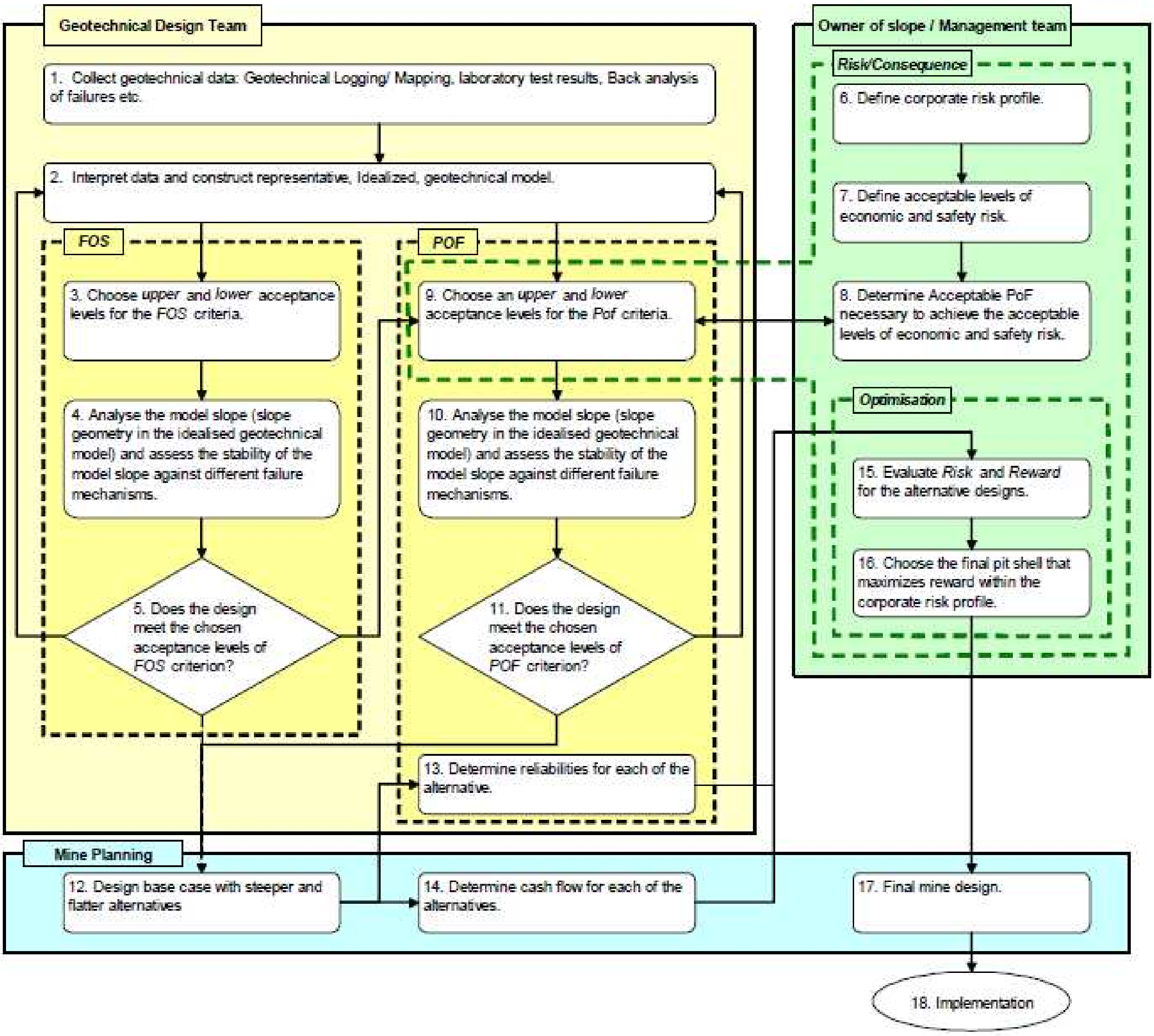


Figure 2-16: Generic flow chart for open pit slope design (After Steffen et al. 2008)

One of the objectives of the risk analysis is the evaluation of slope failure on safety of personnel (SRK, 2006). The evaluation is completed by estimating the probability of fatality caused by slope failures and then comparing this result with acceptability criteria (Steffen et al. 2007). The calculated probability of fatality values would need to be compared with benchmark criteria. A typical graph used for this purpose is indicated in Figure 2-17, which represents acceptability criteria for fatalities defined by different sources (Tapia et al. 2007). The areas in the graph related to tolerability of risk are derived from risk guidelines developed in various countries and correspond to risk thresholds in terms of local acceptability of deaths from industrial and other accidents (HSE, 2001). It is plotted as the number of fatalities from accidents versus the annual probability of exceeding that number.

Steffen et al. (2008) describe the “Local Tolerability Line” which defines a region which is characterized by both high frequencies and severe consequences (the “Intolerable” region). The region between this line and the “Local Scrutiny Line” is a region of possibly unjustifiable risk. Between this latter line and the “Negligibility Line” is a region which is judged to be tolerable but for which all reasonably practicable steps should be taken to reduce the hazard further. This is the ALARP region (As Low As Reasonably Practicable). All combinations of frequency and number of fatalities which fall below the “Negligibility Line” are considered to be negligible.

The graph also contains fatality criteria developed by the dam engineering discipline, which is of interest as benchmark criteria, because it is based on a conservative approach to risk, since the consequences of dam failures are so devastating. Tapia et al (2007) noted, of particular interest is the single fatality at an annual probability of 10^{-4} identified as “The proposed BC Hydro individual risk”. This number represents the risk exposure to ‘natural death’ by the safest population group in North America aged between 10 to 14 years within a person’s life cycle. The interpretation is that 1 in 10,000 persons between the ages of 10 and 14 years will die every year from natural causes (HSE, 2001). This is also popularly accepted as the boundary between voluntary and involuntary risk. Accepting this as a criterion for slope design, implies, therefore, that a person subjected to the open pit working environment is not exposed to greater death risk resulting from a slope failure than he is of dying due to natural causes (SRK, 2006).

The incorporation of risk analysis into slope designs has some challenges, especially due to the requirement for more input information than the conventional approach (Chiwaye, 2010).

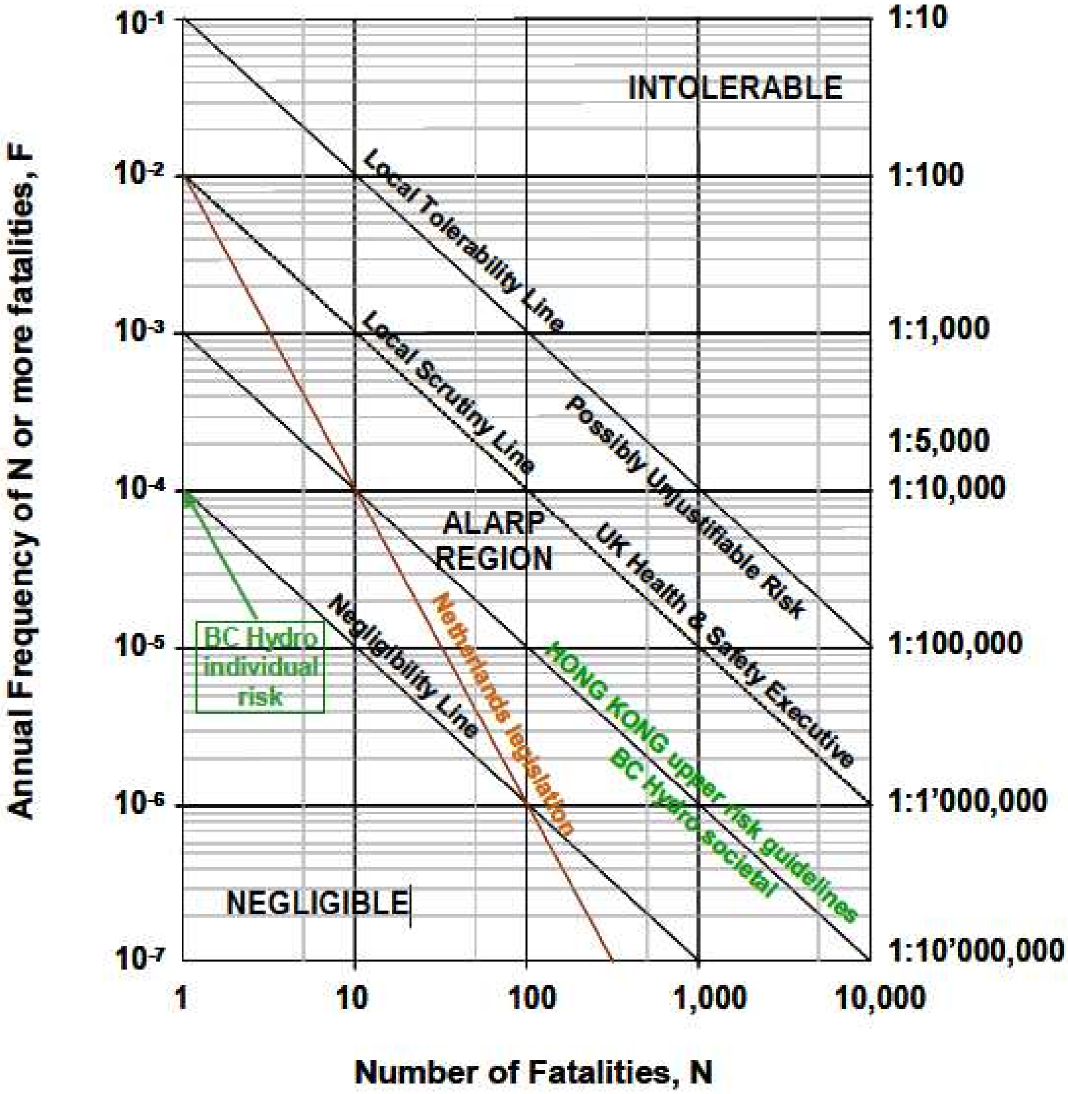


Figure 2-17: Benchmark criteria considering extreme eventualities per annum, used in engineering disciplines (where ALARP = As Low As Reasonably Practicable) (Steffen *et al.,* 2008)

2.3 Literature Review Findings

A review of the literature indicated that classification systems play an integral role in the realm of rock mechanics and rock engineering. The evolution of the systems was also particularly useful to note, as this helped improve the various systems for their intended purpose.

The rock mass classification systems can simply be described as a method that assigns a quantitative rating to a rock mass as opposed to a qualitative term. Laubscher (1977) indicated that classification should be used as a first step protocol, and that each individual case should be examined in more detail.

Rock Mass Classification Systems work most effectively when the user is aware of the limitations of the system and when good quality data is collected. The user should also have practical experience.

In terms of risk a based design approach, the methodology provides for a rational approach to defining the risk at an early stage of the intended project. An outcome of adopting the process is that it ensures that risks to personnel, and of economic impact, equipment damage, force majeure, negative public relations and industrial action are within the criteria set by the mine owners (Steffen et al. 2008). The challenge of adopting this approach has to do with the need for more input information (Chiwaye, 2010). The use of non-formal sources of information is also a requirement for this process (Steffen et al. 2008). It is the view of the author that mining executives are understanding the value in employing the risk based design approach to their environments and that this system is gaining momentum.

Based on the research findings, a geotechnical investigation and a consequence based rock fall assessment was completed for B2Gold Namibia's Otjikoto Gold Mine using Laubscher's MRMR. Chapter 3, which follows, details the geotechnical investigation. This is followed by the rock mass characterisation undertaken which is presented in chapter 4. Laubscher's MRMR system was selected as the system going forward as the author has the most experience with this system. As pointed out in Chapter 2, experience with the classification system to be used accounts for more reliable results.

3 GEOTECHNICAL INVESTIGATION

In 2012, B2Gold requested that a geotechnical assessment of sub-surface ground conditions of the footprint of Otjikoto gold mine project be conducted for the design of the optimum final designed slope angles for an open pit mine.

3.1 Regional Geological Setting

The Otjikoto Gold Project is set within the northeast-trending, intracratonic arm of the Damaran Orogen. Figure 3-1 illustrates the major tectonostratigraphic zones of the Damaran Orogen.

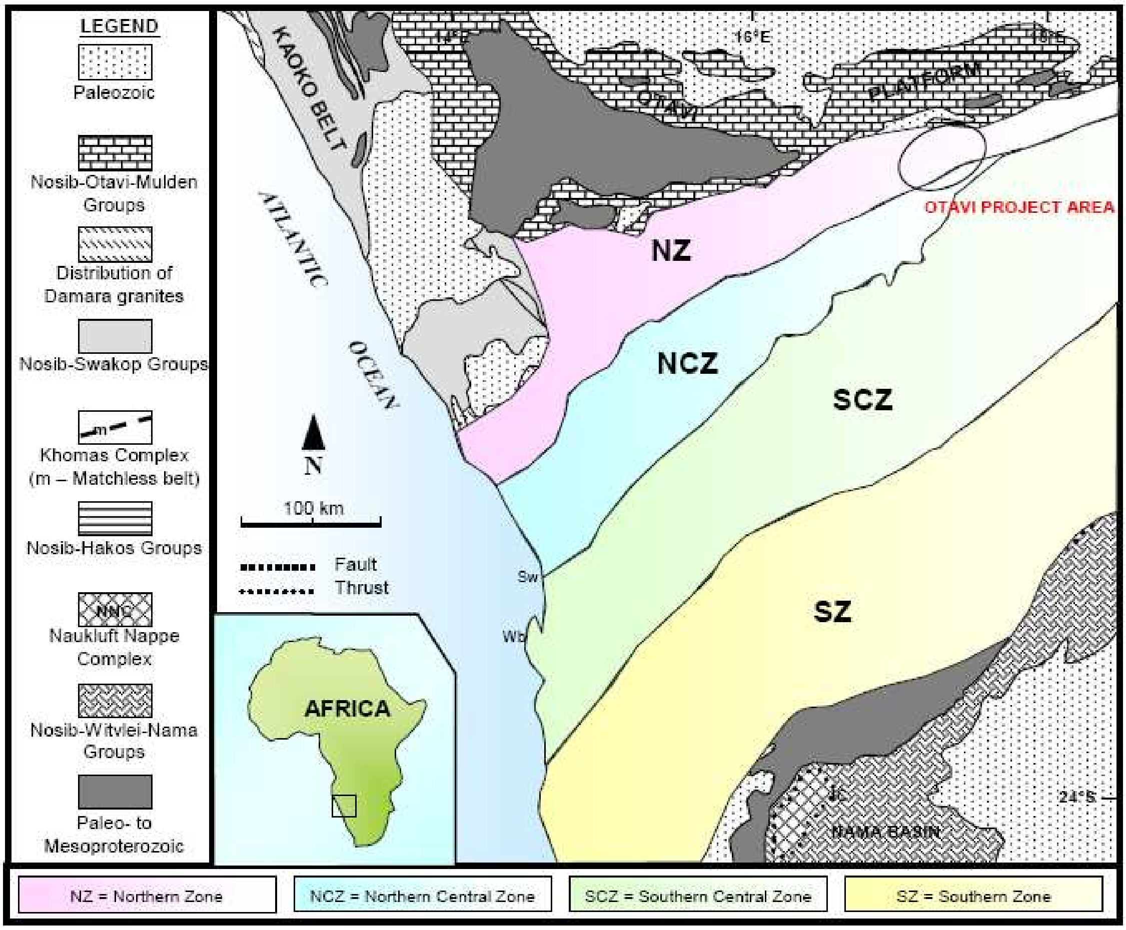


Figure 3-1: Major Tectonostratigraphic zones of the Damaran Orogen (Miller, 1983)

The regional area is presumed to traverse the Northern Central Zone ("NCZ") and the Northern Zone ("NZ") of the Damaran tectonostratigraphic zones with the properties lying predominantly within the "NZ" as shown in Figure 3-1.

The region is covered by the Nosib Group, which comprises fluvial sedimentation, the Swakop Group, comprising pelites and carbonate rocks which are found in the NZ and CZ. The NZ comprises carbonate deposits which form the Otavi Group with stromatalite deposits that extend over hundreds of kilometres (van der Merwe and Jones, 2005).

D1 recumbent folding was followed by intrusion of granitic rocks, uplift and erosion in the coastal arm and deposition of molasses (the Mulden Group) in the Northern Platform region. The D2 deformational event has resulted in partly overturned folding and refolding of the Otjiwarongo-Otavi region and was followed by intrusion of granites. D3 doming is the final major deformational event in the CZ (Miller, 1983b) and resulted in south-easterly directed thrusting. D3 was followed by the intrusion of the

Donkerhuk Granite. Isostatic uplift and rise of isotherms in the CZ caused melting of both the basement and cover rocks in the SZ and the emplacement of post-tectonic intrusions (SRK, 2010).

The Damaran succession above the Nosib Group can be clearly differentiated into a “platform facies” and a “basin facies”. The “platform facies” sequence comprises the Tsumeb Subgroup, Chous Formation and Abenab Subgroup of the Otavi Group.

The Abenab Subgroup consists of dolomite and limestone with subordinate shale, arkose and greywacke. The Chuos Formation of the Tsumeb Subgroup can often be clearly distinguished photo-geologically in outcrop, and comprises predominantly a variety of other rock types, such as pebbly schist, quartz-mica schist, quartzite, conglomerate, dolomite, iron-formation, calcsilicate rocks, etc (SRK, 2010).

The remainder of the Tsumeb Subgroup, above the Chuos Formation, is reported to consist of thinly bedded or massive dolomite and limestone with occasional chert beds, intraformational breccias or conglomerates and stromatolitic horizons with minor shale near the top of the succession. The Mulden Group unconformably overlies the Tsumeb Subgroup and represents a molasses assemblage (SRK, 2010)

The “basin facies” of the middle Damaran comprises the Kuiseb and Karibib Formations of the Khomas Subgroup. The regional geology of the Otavi Exploration Project is shown in Figure 3-2. Intrusions into the Damaran in the Otjiwarongo-Otavi region appear to be overwhelmingly granitic in composition, grouped into syntectonic granites and post-tectonic granites (van der Merwe and Jones, 2005).

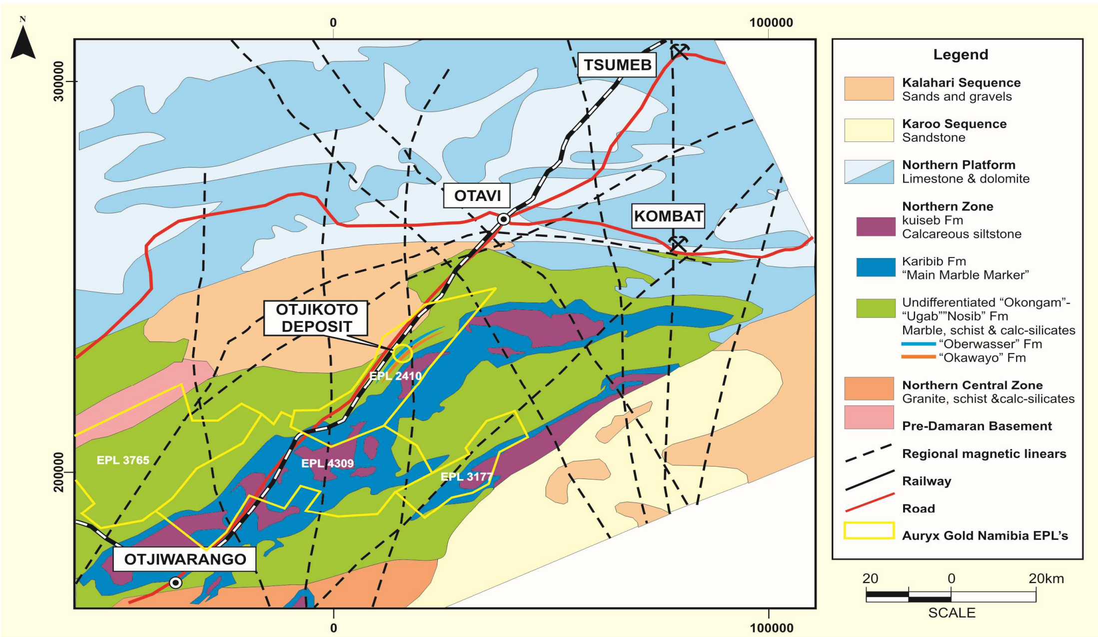


Figure 3-2: Regional Geology of the Otavi Exploration Area (SRK, 2010)

3.2 Local and Property Geology

The current geological understanding of the area has been determined by the Geographical Information System (GIS) compilation of photo-geological studies, outcrop mapping, geophysical interpretations and drillhole data. The area can be divided into eight geological units as listed in Table 3-1:

Table 3-1: Geological unit with associated geological codes

Geological Unit	Geological Code
Albitite	ALB
Biotite Schist	BS
Calcrete	CALC
Footwall Marble	FW
Hornfels	HF
OTB Marble	OTB
Schist	SCH
Weathered Albitite Schist	WAS

The Karibib Marble represents a marker horizon over much of the central parts of the project area. In most places where it was observed in the field it consists of a continuous, thick succession of white to light grey marbles which are usually quite well bedded. The "Main Marker Marble" often occupies prominent ridges with excellent outcrop. The Karibib Marble is folded in an open, synformal structure immediately to the east of the deposit (van der Merwe and Jones, 2005).

Below this main marker horizon the Karibib Formation is seen to include thick units of dark grey marble, schists (including biotite schists and graphitic schists) as well as calc-silicate horizons which form part of the Kuiseb Formation and Khomas Subgroup. It occupies flat low lying areas in the axis areas of the Main Marker Marble synclines. Kuiseb Formation calcareous siltstones occupy the core of the main synforms in the area. The Otjikoto Project is hosted within extensively albitised schist of the basal Oberwasser and Okawayo Formations (van der Merwe and Jones, 2005).

The gold mineralisation is hosted by a series of thin (<10 cm) sheeted veins, which lie essentially parallel to the northeast trending foliation of the schists and granofels (metamarls) of the basal Oberwasser Formation*.* The lithotypes at the Otjikoto gold deposit have been divided into three lithostratigraphic units: from bottom to top the OTA fels, the OTB marble and the OTC albitite-hornfels. The OTC unit hosts the mineralised vein system and is underlain by the 6 m to 10 m thick unmineralised OTB calcareous marble and the OTC albite hornfels. The mineralised veins are essentially unmetamorphosed and relatively undeformed (SRK, 2010).

3.3 Weathering

The depth of the weathering profile in the vicinity of the final pit slope is variable and comprises a highly weathered profile to a depth of between 20m to 30m below ground level, underlain by a transition zone comprising moderately weathered albitite schists to a depth of approximately 50m. The average weathering profiles are 50m for the eastern slope and 41m for the western slope. For mine design purposes the cut off for the weathered profile was considered to be at 40m below ground level.

3.4 Regional Geological Structure

Structurally most of the faults so far discovered and mapped in the Damaran of the Otjiwarongo-Otavi Region appear closely related to the D2 deformational event.

A number of closely spaced strike slip faults were interpreted photogeologically to follow the southern (possibly overturned) flank of the major D2 anticline to the south of the Otavi Valley Synclinorium (SRK, 2010)

The Waterberg Fault essentially forms the south-eastern boundary of the project area and does not intersect the proposed pit. It is defined as a reverse fault, upthrown on the north-western side. The east-west fault on farm Warlencourt 99 and other faults may be splays off the main Waterberg Fault (SRK, 2010).

Mapped dykes are rare and probably mainly related to Karoo alkaline intrusives. Exceptions are the two east-west trending dyke-like features in the Kuiseb Formation.

The structural model is a fault map interpreted mainly from drillhole data and geophysics by the Otjikoto exploration team. Due to the calcrete cap photo interpretations are not possible. The Otjikoto structural model is presented as Figure 3-3 where dark blue lines indicate faults which were identified as strong aquifers, light blue indicates week aquifer faults and black lines indicate other faults.

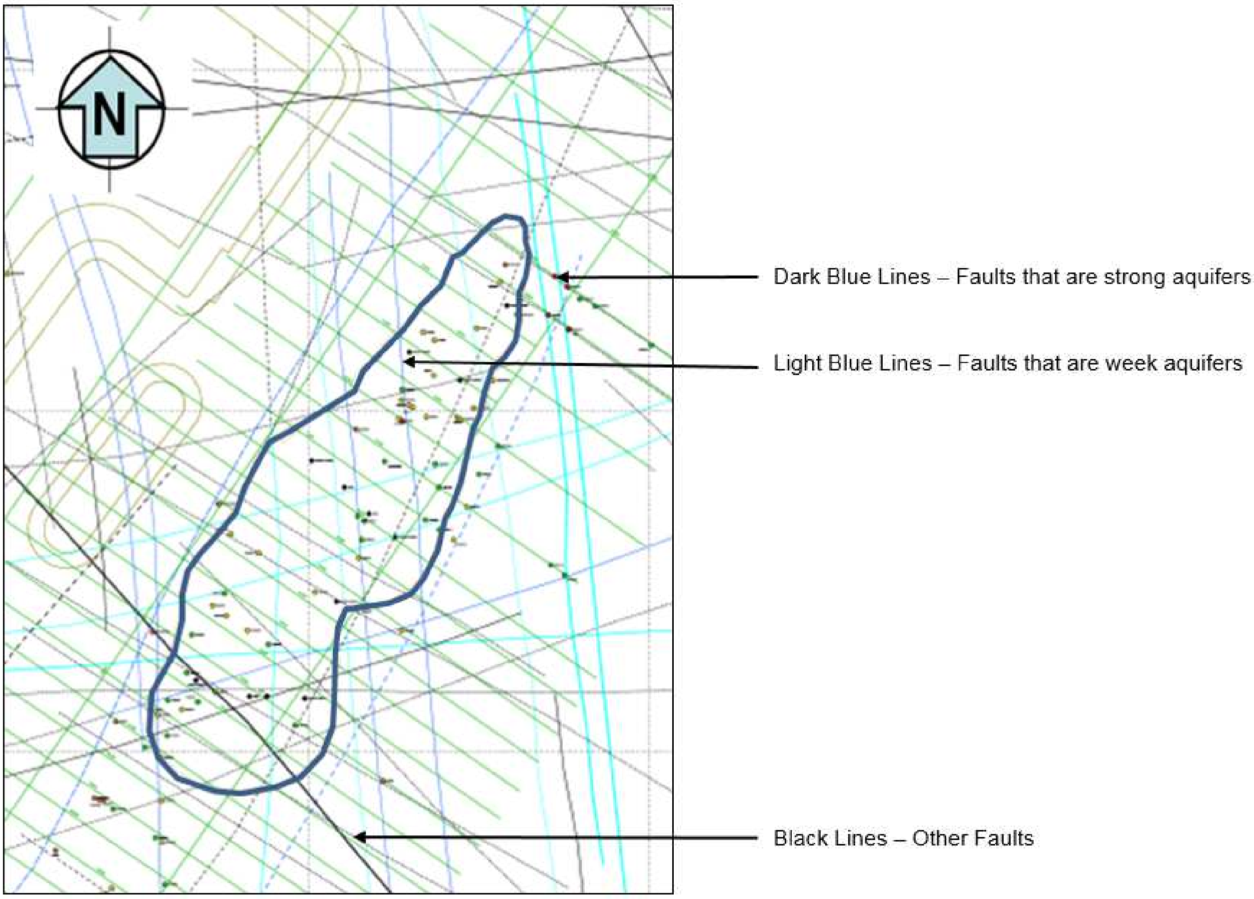


Figure 3-3: Interpreted fault map

3.5 Geotechnical Drilling

Core from the 2012 drilling campaign was geotechnically logged on site as it was recovered. Parameters recorded were in accordance with the Laubscher (1990) Mining Rock Mass Rating System (MRMR). Core logging to date constitutes the holes as detailed in Table 3-2. A plan view of the proposed pit with the locations of the 2012 drillhole collars and section lines for the analysis is presented as Figure 3-4.

Table 3-2: Core logging carried out on the drillholes as shown

Hole ID	Location X	Location Y	Location Z	Azimuth (°)	Dip (°)	Drillhole Depth (m)
OTGT12-005	720255	7788628	1507	300	-60	170.88
OTGT12-006	720009	7788853	1505	090	-60	119.86
OTGT12-007	720296	7789171	1505	315	-60	119.86
OTGT12-008	720445	7789089	1505	135	-60	149.90
OTGT12-009	720583	7789308	1505	315	-60	129.88
OTGT12-010	720082	7788439	1505	115	-60	185.79
OTGT12-011	719989	7788155	1505	305	-60	212.67
OTGT12-012	719670	7788208	1505	115	-60	212.66
GT1	720105	7788774	1504	315	-60	199.78
GT2	720177	7788697	1506	135	-60	200.00
GT3	719827	7788160	1507	270	-60	247.79
GT4	719877	7788161	1508	090	-60	249.00

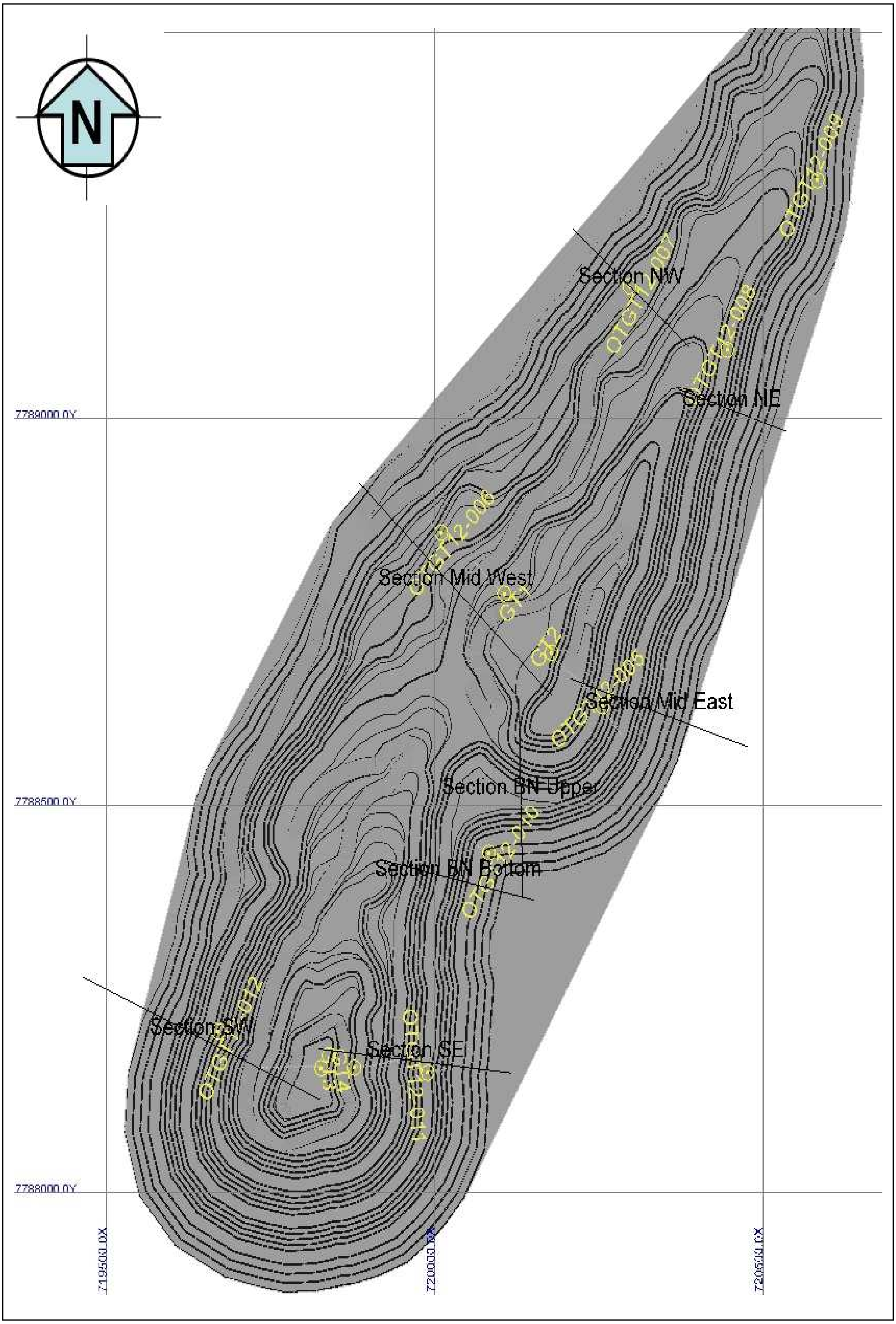


Figure 3-4: Plan view showing drillhole locations and section lines

3.6 Geotechnical Logging

The cores recovered from the geotechnical drillholes were geotechnically logged by the author, according to recommended methodologies. The adopted method ensured that geotechnical zones were identified effectively. Geotechnical zones are zones of rock that are 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 Quality Designation (RQD);
- The rock types present along the depth of the hole;
- The quality of matrix / rock mass defects (faults, intense fracturing, shear zones and zones of deformable material)
- The Intact Rock Strength / hardness (IRS)
- The quantity of solid rock versus weak rock
- The total number / density / frequency of structures (FF)
- The degree and nature of rock weathering
- The condition of structures, (wall alteration, infilling, roughness profile)

3.7 Structural Evaluation

The 2012 geotechnical drilling campaign made use of the Reflex ACT II orientation technique for core orientation purposes. Open joints were measured and recorded in the field with a goniometer and closed joints were measured using the software program Stereocore.

Drillholes GT1, GT2, GT3 and GT4 were logged with an Optical Televiewer and the results obtained were incorporated into the study.

Orientation data from the drillholes have been analysed in the software program DIPS version 5.0, using Fischer's method, equal area and lower hemisphere stereographic projections.

3.7.1 Northern Sector

Joints measured from the Northern Sector, drillholes OTGT12-005 to OTGT12-009, are presented as Figure 3-5 and the joint sets identified in Table 3-3.

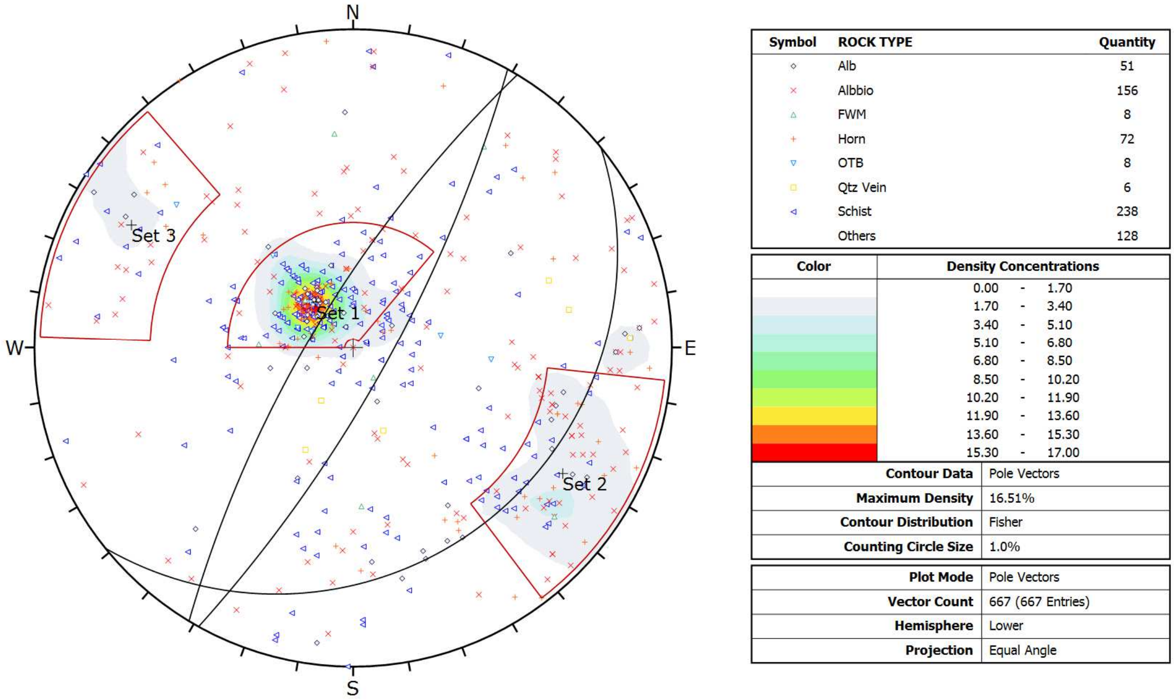


Figure 3-5: DIPS equal area, lower hemisphere stereonet plot showing jointing in the Northern Sector.

Table 3-3: **Average joint set orientations for Northern Sector**

Set	Dip	Dip Direction
1	21	141
2	75	301
3	77	119

Figure 3-6 is a representation of jointing with down hole depth. This indicates a clear concentration of joints from the 40 m to 140 m interval. Joint set 1 is present throughout the drillholes and joint sets 2 and 3 are fairly ubiquitous.

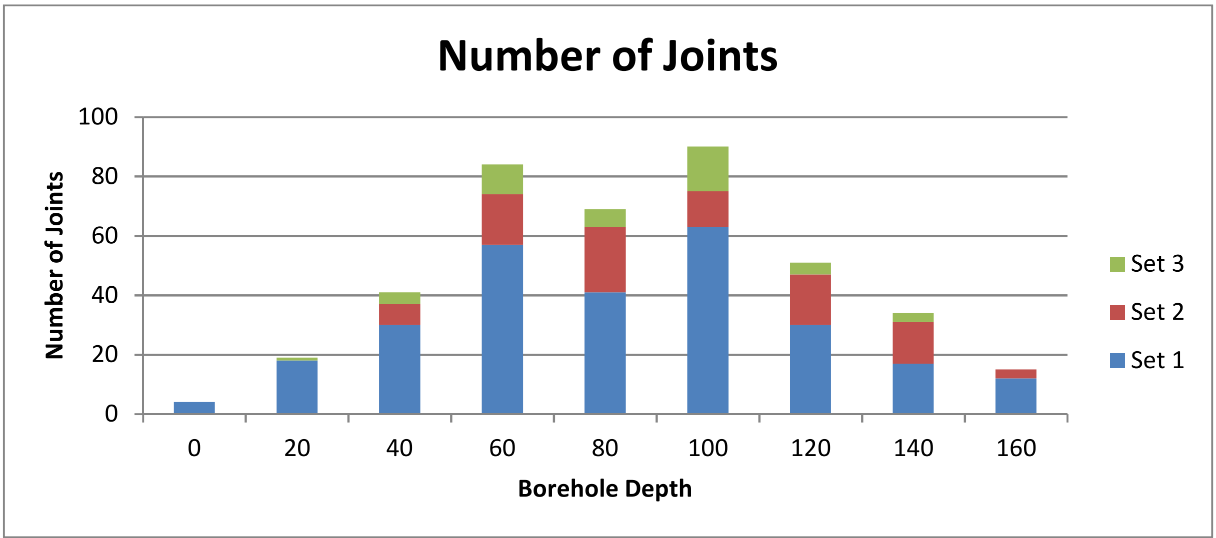


Figure 3-6: Distribution of joints down drillholes OTGT12-005 to OTGT12-009

Distribution of the adjusted joint spacing is presented as Figure 3-7. The joints are generally widely to extremely widely spaced, with joint set 1 displaying joints that are predominantly closely to very widely spaced in nature.

The joint roughnesses for each set are presented as Figure 3-8. Joints are predominantly *rough undulating,* with relatively smaller proportions of *rough stepped/irregular and rough planar* in all sets.

The joint infill for each set is presented in Figure 3-9 and summarised as follows:

- Joint set 1 has a dominant *no infill/staining* (81%) with 11% of joints having *soft sheared fine* material and 6% having a *non-softening fine infill* and the remainder being *non-softening medium* infill
- Joint set 2 has a dominant *no infill/staining* (82%) with 2% of joints having *soft sheared fine* material and 11% having a *non-softening fine infill* and the remainder being *non-softening medium infill*
- Joint set 3 has a dominant *no infill/staining* (77%) with 20% of joints having *soft sheared fine* material and 3% having a *non-softening fine infill*

Extremely Close	Very Close	Close	Moderate	Wide	Very Wide	Extremely Wide
0 m - 0.02 m	0.02 m - 0.06 m	0.06 m - 0.2 m	0.2 m - 0.6 m	0.6 m - 2 m	2 m - >6 m	> 6 m

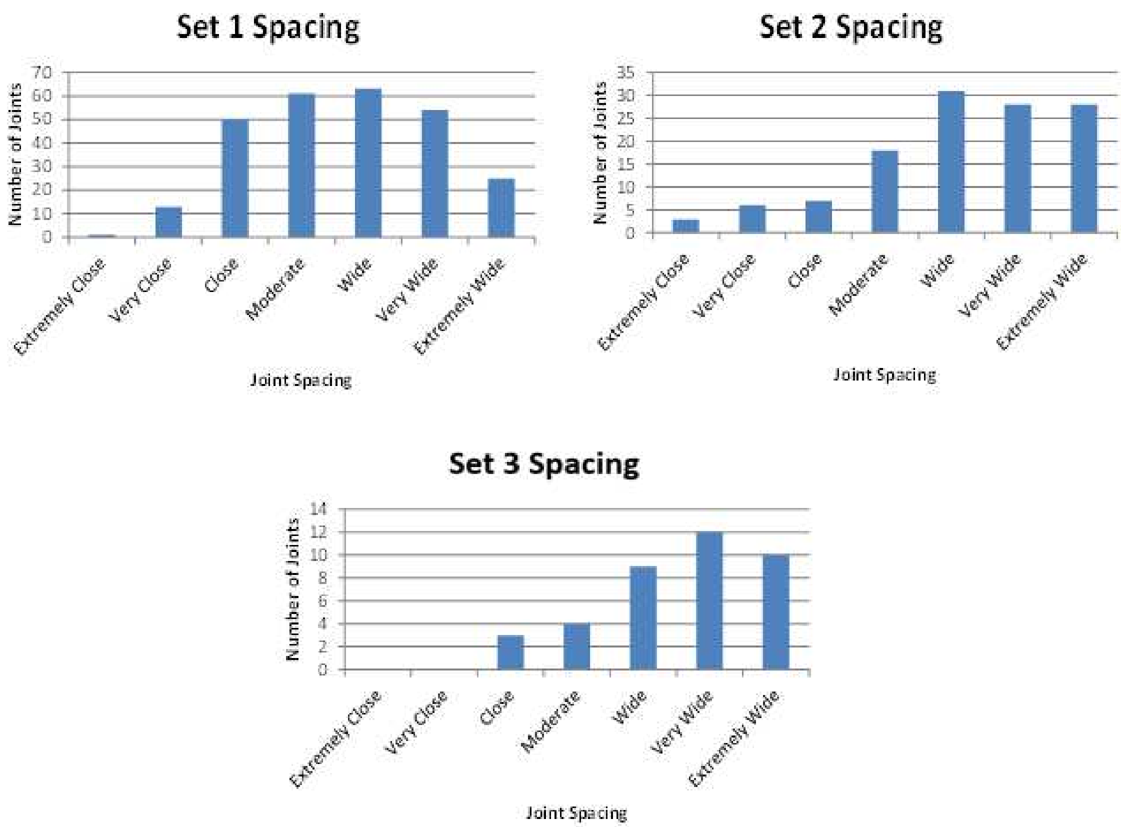


Figure 3-7: Distribution of joint spacing for drillholes OTGT12-005 to OTGT12-009

Figure 3-8: Joint roughness for each joint set (Northern Sector)

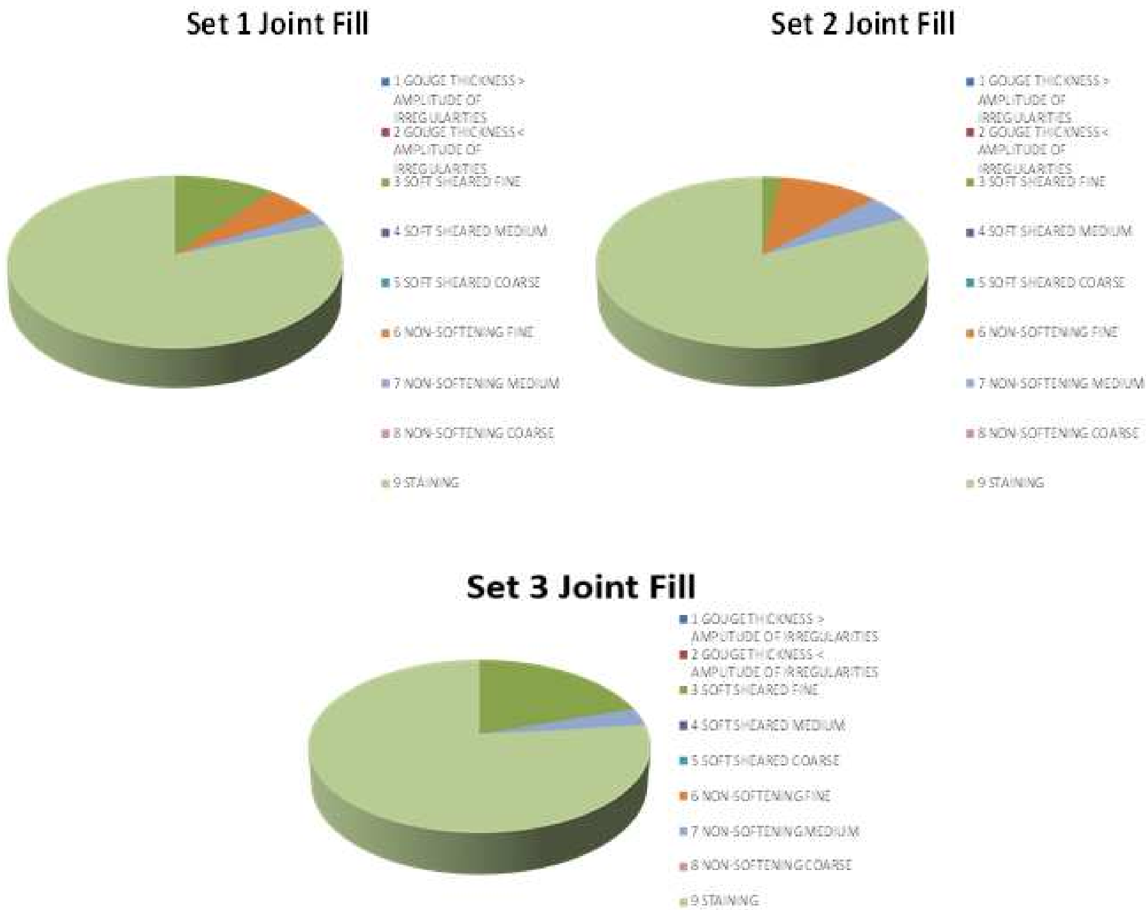


Figure 3-9: Joint fill for each joint set (Northern Sector)

3.7.2 Southern Sector

Joints measured from the Southern Sector, drillholes OTGT12-010 to OTGT12-012, are presented as Figure 3-10 and the identified joint sets in Table 3-4.

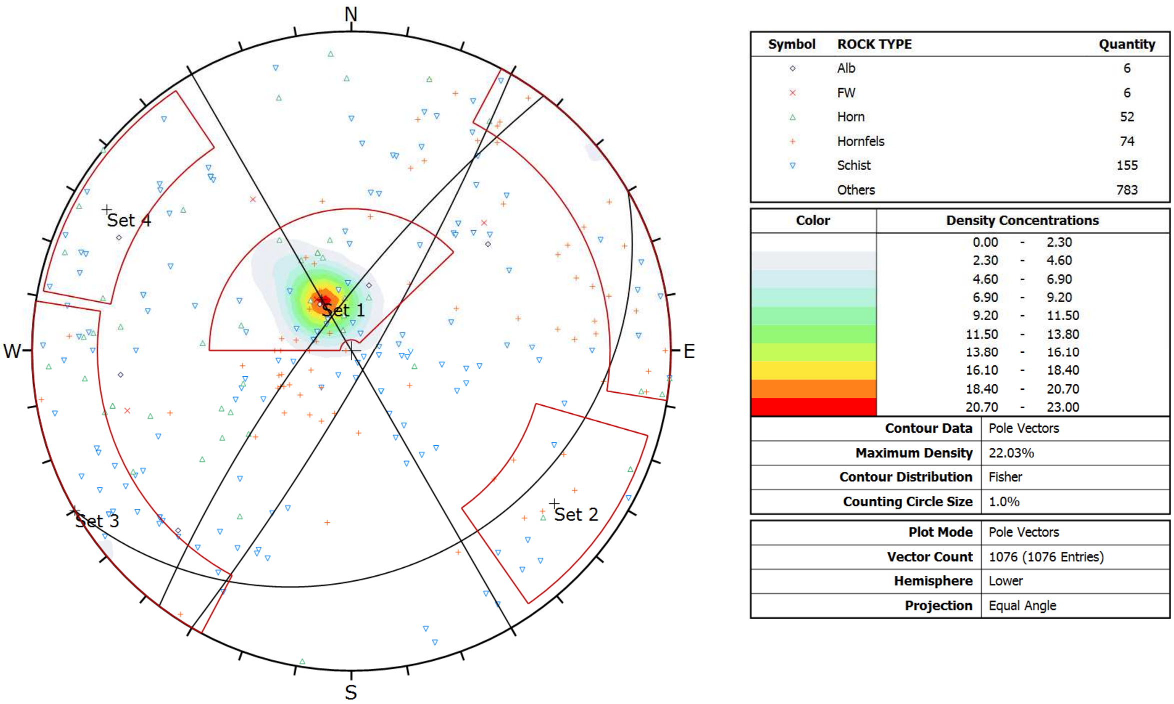


Figure 3-10: DIPS equal area, lower hemisphere stereonet plot showing jointing in the Southern Sector.

Table 3-4: Average joint set orientations for Southern Sector

Set	Dip	Dip Direction
1	21	150
2	77	307
3	90	060
4	83	120

Figure 3-11 is a representation of jointing with down hole depth. This indicates a clear concentration of joints from the 30 m to 220 m interval. Joint sets 1 and 2 are present throughout the drillholes and joint sets 3 and 4 are fairly ubiquitous.

Distribution of the adjusted joint spacing is presented as Figure 3-12. The joints are very variable in nature with joint set 1 having very close to widely spaced joints and joint sets 2 to 4 are predominantly widely to extremely widely spaced.

The joint roughnesses for each set are presented as Figure 3-13. Joints are *rough undulating* for sets 1, 3 and 4, with joint set 2 predominantly being *rough undulating*, with relatively smaller proportions of *rough stepped/irregular, rough planar and smooth undulating*.

The joint infill for each set is presented in Figure 3-14 and summarised as follows:

- Joint set 1 has a dominant *no infill/staining* (96%) with 2% of joints having *soft sheared fine* material *and 2%* being *non-softening medium* infill
- Joint set 2 has a dominant *no infill/staining* (83%) with 1% of joints having *soft sheared fine* material *and 14%* having a *non-softening fine infill* and the remainder being *non-softening medium* infill
- Joint set 3 has a dominant *no infill/staining* (95%) with 2% of joints having *soft sheared fine* material *and 4%* having a *non-softening medium infill*
- Joint set 4 has a dominant *no infill/staining* (92%) and 8% of joints have a *soft sheared fine* material

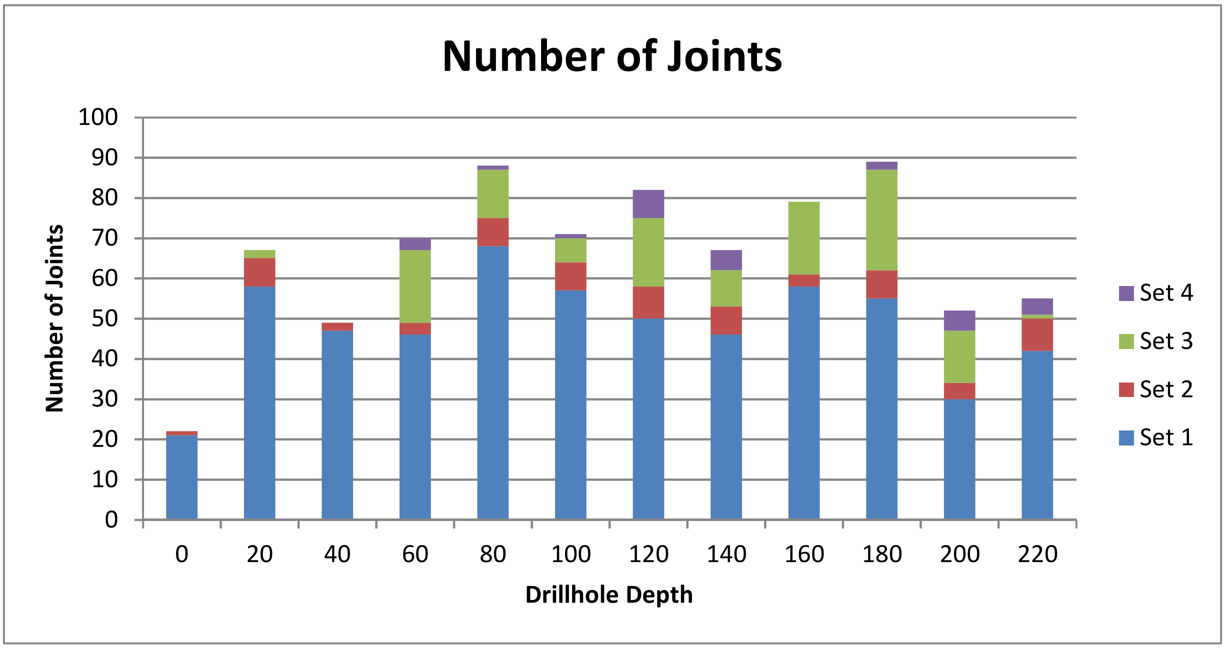


Figure 3-11: Distribution of joints down drillholes OTGT12-010 to OTGT12-012

Extremely Close	Very Close	Close	Moderate	Wide	Very Wide	Extremely Wide
0 m - 0.02 m	0.02 m - 0.06 m	0.06 m - 0.2 m	0.2 m - 0.6 m	0.6 m - 2 m	2 m - >6 m	> 6 m

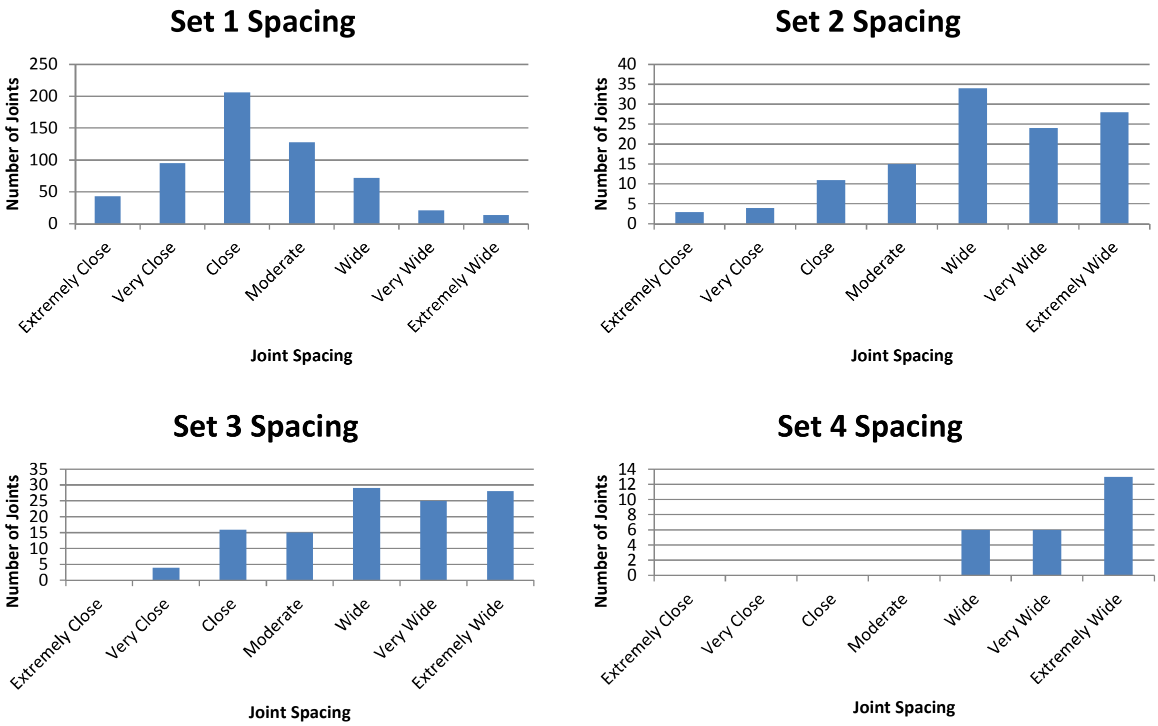


Figure 3-12: Distribution of joint spacing for drillholes OTGT12-010 to OTGT12-012

Figure 3-13: Joint roughness for each joint set (Southern Sector)

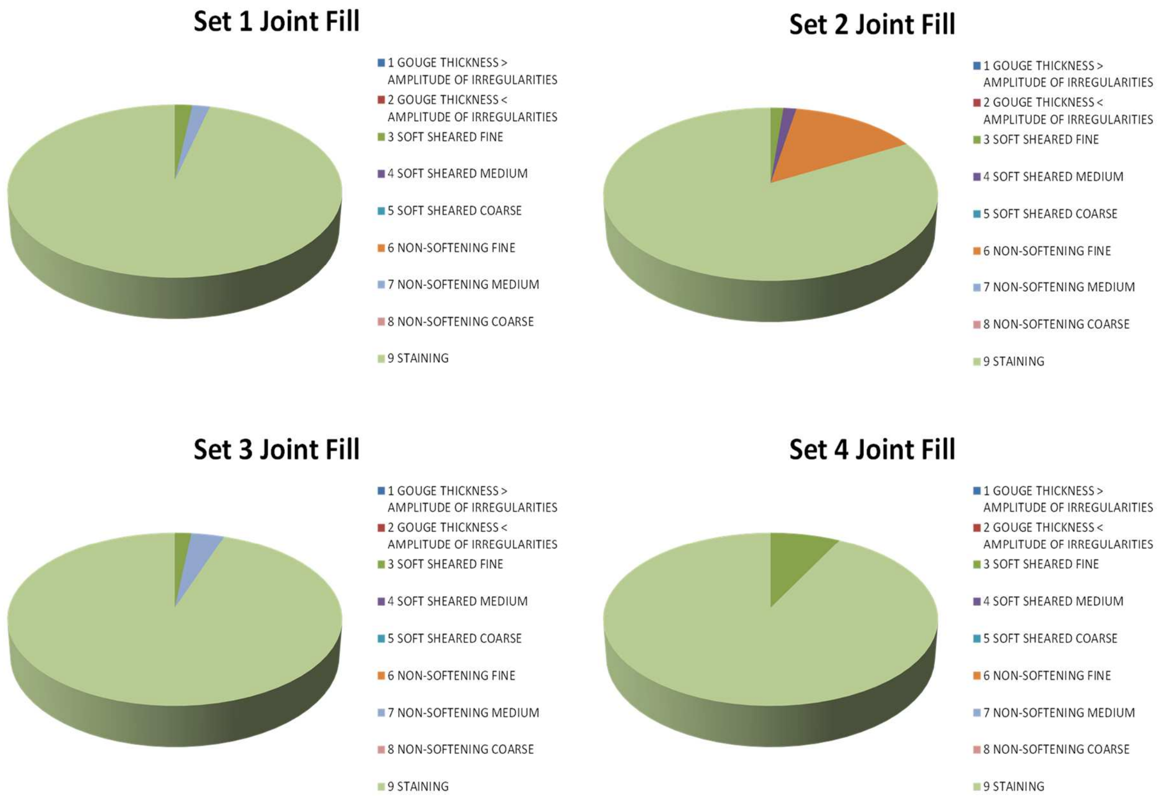


Figure 3-14: Joint fill for each joint set (Southern Sector)

4 ROCK MASS CHARACTERISATION

To classify the quality of the rock mass, use was made of Laubscher's (1990) Mining Rock Mass rock mass classification system. The Laubscher (1990) system was selected as the author has the most experience utilising the system.

4.1 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.

Eight drillholes were geotechnically logged and the data was added to the current geotechnical database for the characterization. The drillhole data includes the identification of the geotechnical interval, the rock type unit, and the values of intact rock strength (IRS) and hardness ratings, RMR_{L90}, fracture frequency rating (FFr), vein formation and joint condition rating (JCr). The average RMR_{L90} results are shown in Table 4-1. The full table of results is also attached as Appendix A.

Table 4-1: Average RMR_{L90} results per geotechnical unit

RMR L90	ALB	BS	Calc	FW	HF	OTB	SCH	WAS
n	46	28	20	5	113	8	134	72
Average	63	69	52	74	59	64	60	47
Avg - Stdv	51	56	45	67	51	56	50	39
Avg + Stdv	75	83	59	81	68	72	71	54
St Dev	12	13	7	7	9	8	11	8
CV	0.20	0.19	0.14	0.09	0.14	0.13	0.18	0.16

4.2 Laboratory Testing Programme

A laboratory testing programme was commissioned with representative samples collected and tested to investigate the geomechanical properties of the various rock units at the site for rock mass classification purposes. The testing programme was carried out by RockLab in Pretoria, South Africa and included the following tests to fill the gaps in the previous testing database:

- 11 Uniaxial Compressive Strength (UCMS) tests, including Young's Modulus and Poisson's Ratio determination
- 57 Uniaxial Compressive Strength (UCSS) tests
- 108 Triaxial Compressive Strength (TCS) tests
- 11 Base Friction Angle (BFA) tests
- 11 Direct Tensile Strength (DTS) tests

The detailed laboratory testing results supplied by RockLab are included in Appendix B with a summary of the laboratory tests which incorporates results from the previous laboratory testing programmes included as Table 4-2 and Figure 4-1.

Table 4-2: **Summary of the laboratory test results**

Rock Unit		**ALB**	**BS**	**Calc**	**FW**	**HF**	**OTB**	**SCH**	**WAS**
Density (g/cm³)	Number of Tests	41	39	13	40	36	44	41	36
	Minimum	2.17	2.25	1.84	2.70	2.68	2.42	2.51	2.15
	Maximum	2.80	2.84	2.67	2.98	5.01	2.80	2.91	2.68
	Average	**2.66**	**2.73**	**2.40**	**2.72**	**2.84**	**2.71**	**2.74**	**2.52**
	Standard Deviation	0.17	0.12	0.21	0.04	0.51	0.05	0.06	0.14
UCS (MPa)	Number of Tests	13	15	10	16	11	15	16	9
	Minimum	93	89	29	65	233	66	152	14
	Maximum	198	281	117	104	389	107	342	91
	Average	**150**	**192**	**70**	**77**	**289**	**84**	**239**	**50**
	Standard Deviation	37	64	28	10	51	11	52	24
Young's Modulus (GPa)	Number of Tests	2	3	0	3	3	4	2	1
	Minimum	73	77	-	39	72	33	57	18
	Maximum	79	78	-	44	75	52	78	18
	Average	**76**	**77**	**-**	**41**	**73**	**46**	**68**	**18**
	Standard Deviation	5	1	-	2	2	9	15	-
Poissons Ratio (ν)	Number of Tests	2	3	0	3	3	4	2	1
	Minimum	0.31	0.21	-	0.15	0.22	0.19	0.17	0.11
	Maximum	0.33	0.31	-	0.19	0.26	0.25	0.26	0.11
	Average	**0.32**	**0.25**	**-**	**0.17**	**0.24**	**0.21**	**0.22**	**0.11**
	Standard Deviation	0.02	0.05	-	0.02	0.02	0.03	0.06	-
BFA (°)	Number of Tests	3	3	-	2	4	3	4	3
	Minimum	27	30	-	32	34	34	31	33
	Maximum	35	36	-	41	40	42	33	37
	Average	**32**	**32**	**-**	**37**	**37**	**39**	**32**	**35**
	Standard Deviation	4	4	-	6	3	4	0	2
Direct Tensile (MPa)	Number of Tests	2	3	0	0	3	0	3	0
	Minimum	6	6	-	-	5	-	7	-
	Maximum	26	13	-	-	13	-	23	-
	Average	**16**	**9**	**-**	**-**	**9**	**-**	**15**	**-**
	Standard Deviation	14	4	-	-	4	-	8	-

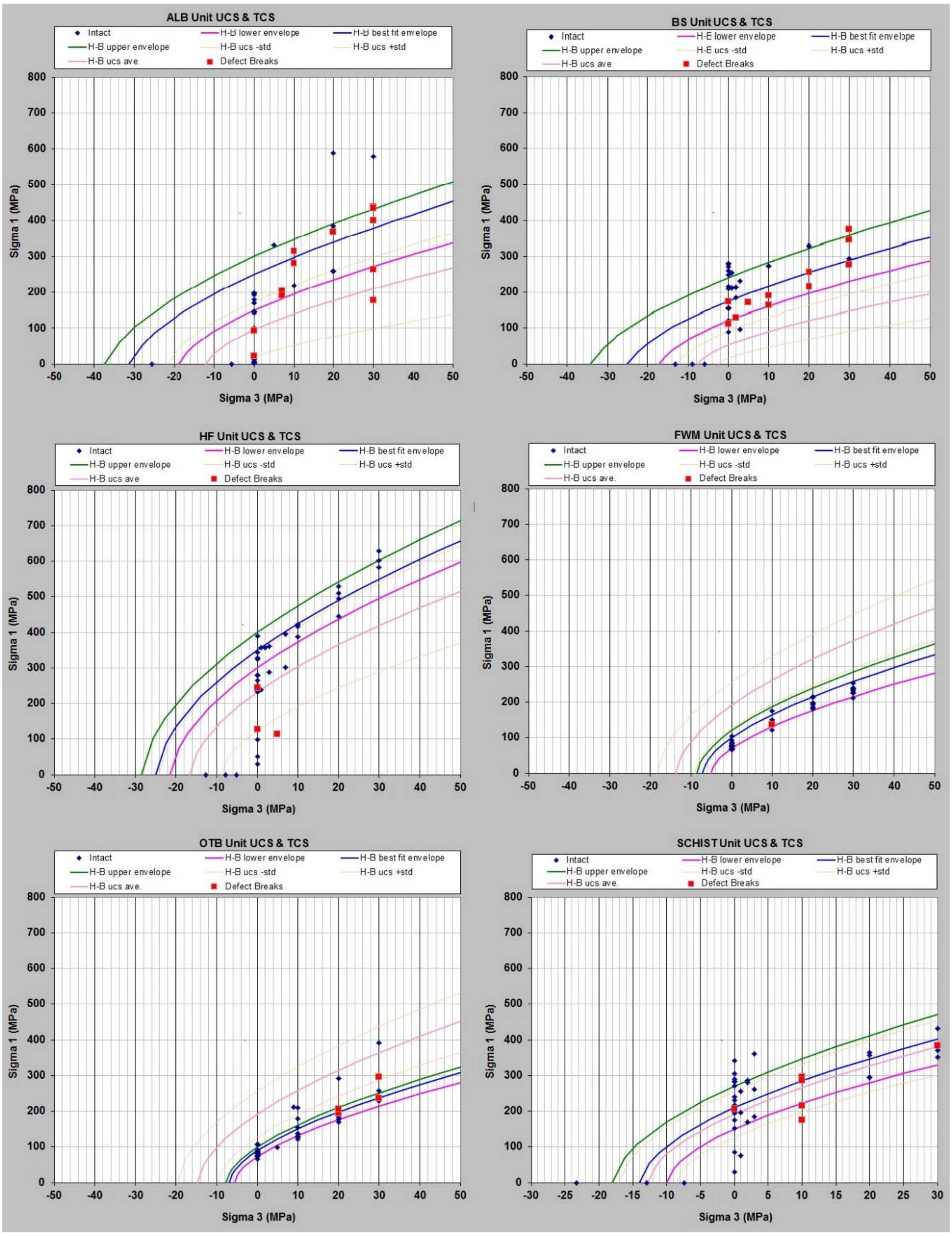


Figure 4-1: Summary of laboratory tests

4.2.1 Density and Strength Properties of Intact Rock

The rock unit weight of the various lithologies was derived from the laboratory tests. The corresponding average values and variability characteristics are summarised in Table 4-2. The strength characteristics of the intact rock material were investigated through the execution of UCS and TCS tests and the results of these tests were used for deriving rock mass strength properties.

4.2.2 Elastic Properties of Intact Rock

The elastic properties of the intact rock represented by the Young's Modulus and the Poisson's Ratio were derived from the UCM tests where the measurements of deformation during the tests were recorded utilising strain gauges.

4.3 Geotechnical Investigation Conclusions

From this chapter the following may be concluded:

- Geotechnical logging was undertaken to determine the geotechnical properties of the rock mass
- Rock Units identified from the geotechnical logging include albitite, biotite schist, calcrete, footwall marble, Hornfels, OTB marble, schist and weathered albitite schist.
- The lithologies identified within the vicinity of proposed mine range in intact rock strength from 50 MPa to 289 MPa for the various lithologies. These strength values 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 proposed mine. Chapter 4 details the characterisation of the rock mass using Laubscher's MRMR system.

4.4 Empirical Design Charts

Design charts provide empirical design slope angle and slope height guidelines by relating the knowledge gained from the understood performance of slopes at various mine locations with the results provided by a classification scheme (Lorig et al. 2008). The most widely used empirical design chart tool is the published Haines and Terbrugge (1991) chart. This chart (Figure 4-2) is based on the Laubscher (1990) MRMR rock mass rating system. It must be acknowledged that design charts are a useful tool for predicting preliminary slope angles at a desktop or pre-feasibility level study, but must be used with caution, or as a precursor to more rigorous design approaches at a feasibility or design level study. This current study is at a feasibility level, and as such, the empirical design chart is used to demonstrate the preliminary slope angles one can expect before the more robust analyses are carried out in Chapter 6. The adjustment factors used to determine MRMR from the average RMR values are listed below:

Weathering effects	-	100 percent
Blasting effects	-	94 percent
Stress effects	-	95 percent
Joint Orientations	-	94 percent

The majority of the slope comprises of the rock types Hornfels and Schist. For the purpose of the empirical design, the determined RMR values for these lithologies will be averaged and adjustments for the mining effects as described above will be applied. The result of the study is presented as Table 4-3.

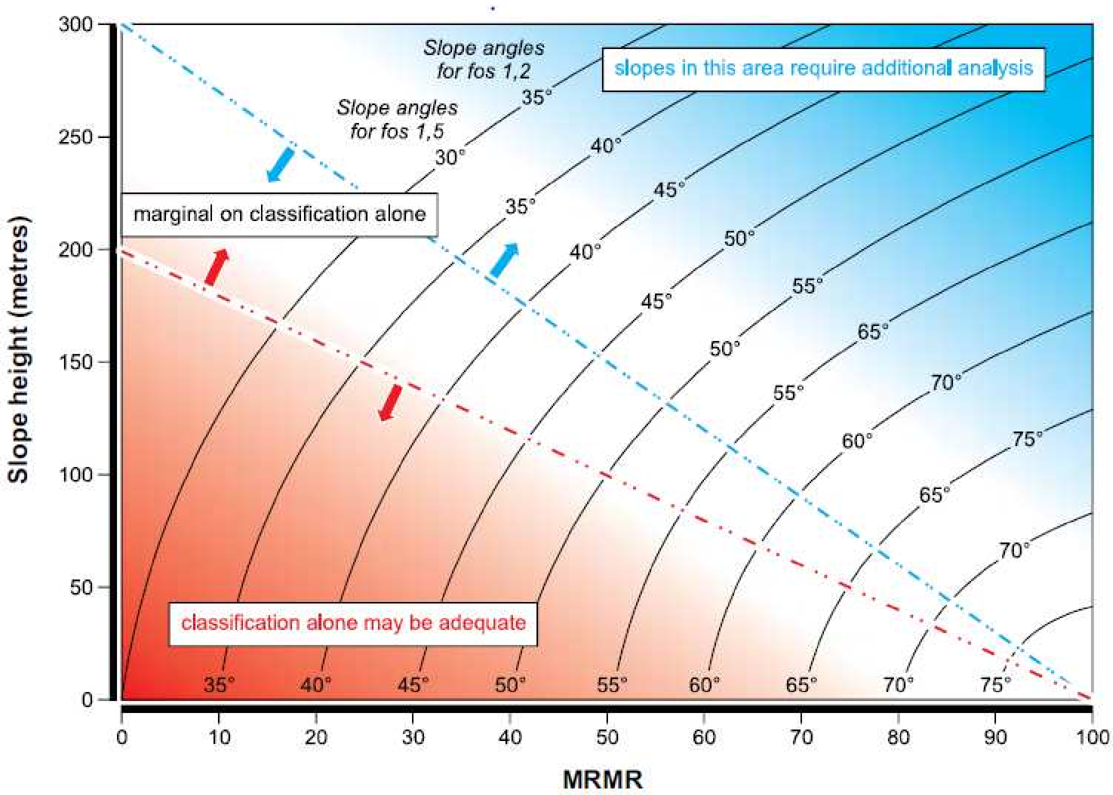


Figure 4-2: Haines and Terbrugge Chart for determining slope height and slope angle (After Haines and Terbrugge, 1991)

Table 4-3: **Summary of the empirically determined slope angles**

Empirical Design Section	**Average MRMR**	**Slope Height**	**Determined Slope Angle (Haines and Terbrugge, 1991)**	
			FOS = 1.5	**FOS = 1.2**
BN Bottom	50	167 m	44.5°	49.5°
BN Upper	50	153 m	45°	50°
Mid East	50	177 m	43°	48°
Mid West	50	171 m	44°	49°
North East	50	124 m	48°	53°
North West	50	115 m	49.5°	54.5°
South East	50	230 m	36°	41°
South West	50	224 m	37.5°	42.5°

From the eight sections analysed, six fell in the additional analyses required area of the graph. Chapter 5 details the determination of the design strength parameters that were used to carry out the additional analyses required to produce feasibility level design slope angles.

5 Design Strength Parameters

5.1 Rock Mass Strength Parameters

The estimation of rock mass strength properties was based on the geotechnical drillhole logs and the results of the laboratory testing programme. The generalised Hoek-Brown failure criterion was directly used in both the limit equilibrium analysis (SLIDE) and the finite element analysis (PHASE2). Equivalent Mohr-Coulomb parameters are calculated for reference and inclusion in anisotropic analyses. The procedure adopted to estimate the Hoek-Brown material constants (m_b, s, a) is illustrated in the diagram presented in Figure 5-1.

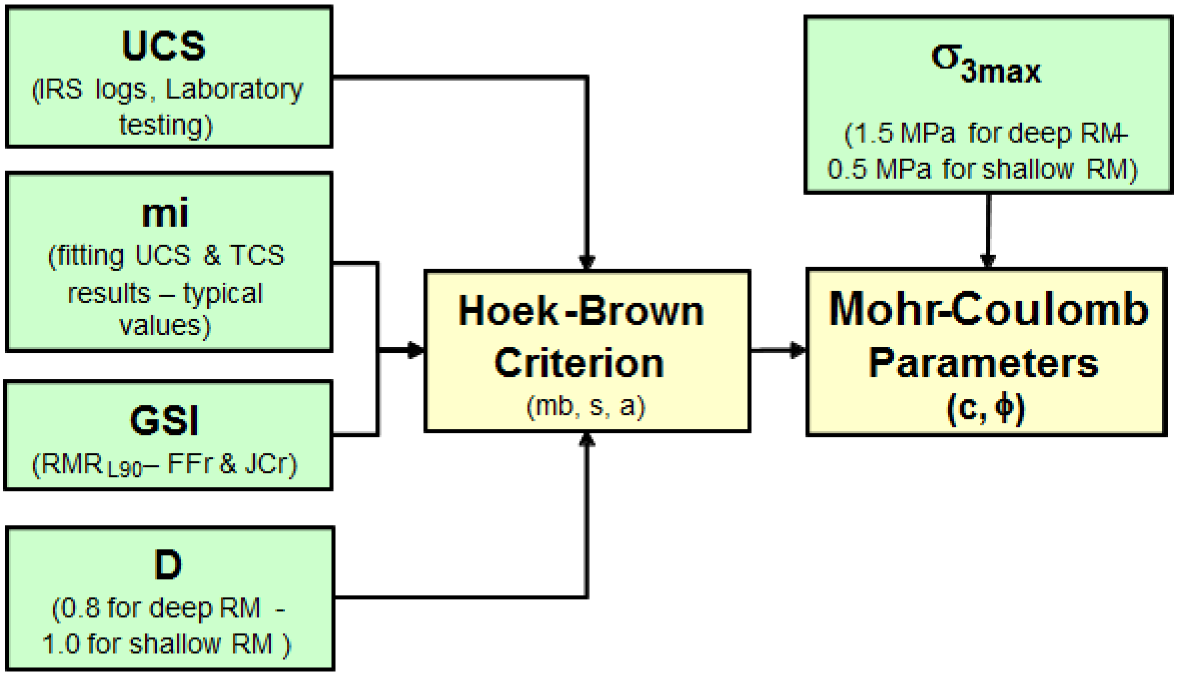


Figure 5-1: Methodology for the estimation of rock mass strength parameters

5.2 Intact Rock Strength

In terms of the UCS input, values of Intact Rock Strength (IRS) from the drillhole logs were combined with the UCS averages from the laboratory testing programme to define representative values of the intact rock strength.

5.3 The Intact Rock Material Constant m_i

The estimation of m_i values was based on typical values published in literature for the various rock types and the results were verified by fitting Hoek-Brown failure envelopes with the results of UCS, DT and triaxial testing (Table 5-1).

Table 5-1: Summary of m_i values

mi Values	ALB	BS	Calc	FW	HF	OTB	SCH	WAS
Determined	8	7	9	14	14	13	15	10
Literature	-	10 ±3	-	9 ±3	19 ±4	9 ±3	10 ±3	10 ±3

5.4 Estimation of GSI

The estimations of the Geological Strength Index (GSI) values were based on a conversion of the Laubscher Joint Condition rating (JCr) and the Fracture Frequency rating (FFr) into the Hoek scale for surface condition and structure of the GSI chart as indicated in Figure 5-2.

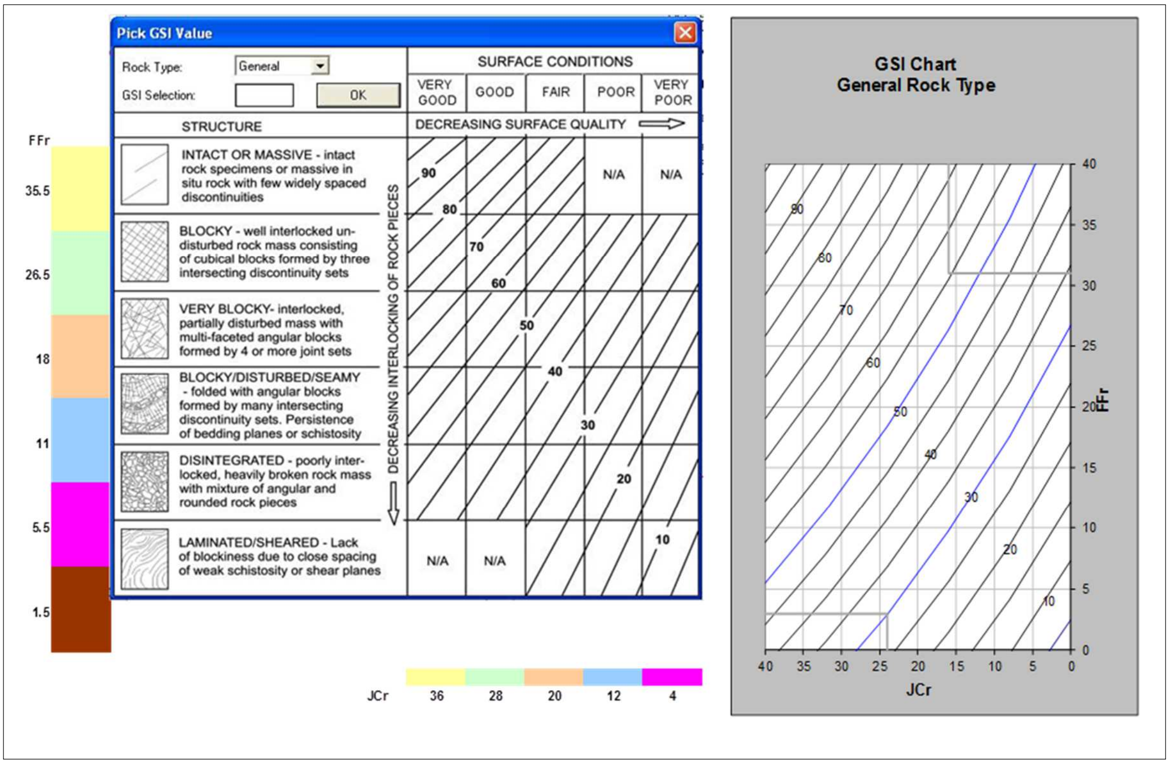


Figure 5-2: **Conversion of the Laubscher FFr and JCr to GSI structure and surface condition**

5.5 Disturbance Factor and Stress Levels

The Disturbance Factor, D is a unit that depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses.

As the rock mass analysis considered deeper seated failure surfaces relatively unaffected by blasting but subject to unloading related relaxation of the rock mass, a D factor of 0.8 was chosen for the estimation of the overall rock mass strength.

The results of the Hoek-Brown rock mass strength parameter calculations are presented as Table 5-2 along with the equivalent Mohr-Coulomb strengths.

Table 5-2: Summary of rock mass strength parameters

No.	Geotechnical Unit Description	Code	H-B Geomechanical Parameters							Equivalent M-C Strength Parameters	
			UCS (MPa)	GSI	mi	D	E (GPa)	ν	s3max (MPa)	c (kPa)	φ (^{0})
1	Albitite	ALB	150	58	8	0.8	10.7	0.32	1.5	1054	47
2	Biotite Schist	BS	192	64	7	0.8	13.5	0.25	1.5	1763	48
3	Calcrete	Calc	70	56	9	0.8	2.6	-	1.5	483	39
4	Footwall Marble	FW	77	69	14	0.8	23.6	0.17	1.5	2085	57
5	Hornfels	HF	289	56	14	0.8	3.9	0.24	1.5	821	50
6	OTB Marble	OTB	84	62	13	0.8	12.2	0.21	1.5	1103	52
7	Schist	SCH	239	55	15	0.8	6.5	0.22	1.5	862	52
8	Weathered Albitite Schist	WAS	50	49	10	0.8	1.2	0.11	1.5	384	36

5.6 Joint Strength Parameters

The Barton-Bandis (B-B) criterion was used to characterize the strength properties of the rock joints and equivalent Mohr-Coulomb (M-C) parameters (c and φ) were estimated for the appropriate level of stress and scale of the slope under consideration. The methodology which was followed is illustrated as Figure 5-3. Key parameters used for the joint strength evaluation were the residual friction angle of the joint RFA (derived from base friction angle laboratory testing), the joint roughness coefficient JRC (from drillhole logs) and the joint compressive strength JCS (from UCS results and assessment of joint alteration in drillhole logs). The resulting joint strength parameters are presented as Table 5-3.

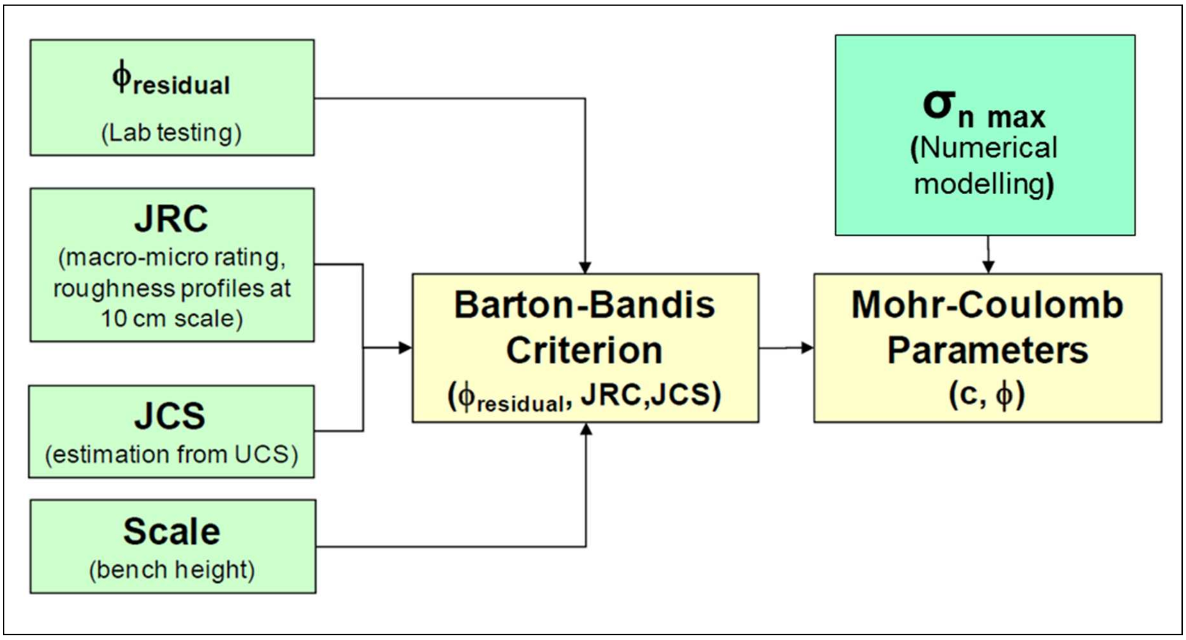


Figure 5-3: Methodology for estimation of joint strength parameters

Table 5-3 shows the parameters that were calculated for the joint network in the model.

Table 5-3: Joint strength parameters

No.	Geotechnical Unit Description	Code	UCS (MPa)	JCS (MPa)	BFA (°)	RFA (°)	JRC log	JCS Field (MPa)	JRC Field	Joints Deep RM (>50 m)	
										c (kPa)	Φ (°)
1	Albitite	ALB	150	120	32	30	6	39	3	16	34
2	Biotite Schist	BS	192	154	32	30	6	50	3	16	35
5	Fw Marble	FW	77	62	37	34	6	20	3	17	37
6	Hornfels	HF	289	231	37	35	6	75	3	19	40
7	OTB Marble	OTB	84	67	39	37	6	22	3	19	40
8	Schist	SCH	239	191	32	31	6	62	3	16	35
9	Weathered Albitite Schist	WAS	50	40	35	33	6	13	3	16	35

5.7 Groundwater Conditions

The groundwater investigation was completed primarily for the sourcing of water, and not for the depressurisation requirements and pore pressure determination within the slope. Based on the results of the groundwater study the initial level of the phreatic surface was assumed to be 40 m below the crest of the slope (phreatic surface level was based on water monitoring drillhole data), with the rebound of the phreatic surface in the vicinity of the pit determined by using the method presented in Figure 5-4. In this method, the following was considered:

The phreatic surface reverts back to its initial level behind the slope at a distance equal to the depth of the pit

The phreatic surface linearly decreases from the initial level to a point 80 m behind the slope at a level corresponding to half the height of the wet portion of the slope

Water pressure is calculated as a function of the gradient of the water surface, as indicated in Figure 5-4 to account for the effect of seepage through the slope. The typical drained model based on this method is illustrated as Figure 5-5 and the undrained model is illustrated as Figure 5-6. The drained model represents the conditions with slope depressurisation measures in place, where the undrained model represents the conditions with no slope depressurisation.

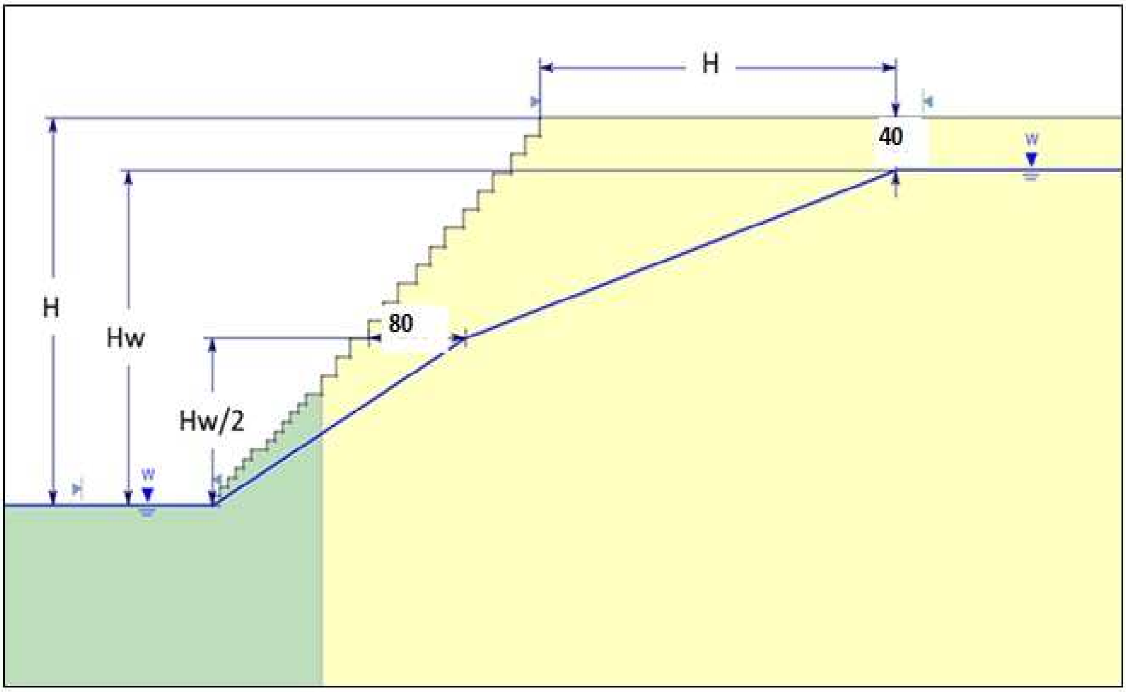


Figure 5-4: Simple model to illustrate groundwater conditions for stability analysis – drained case

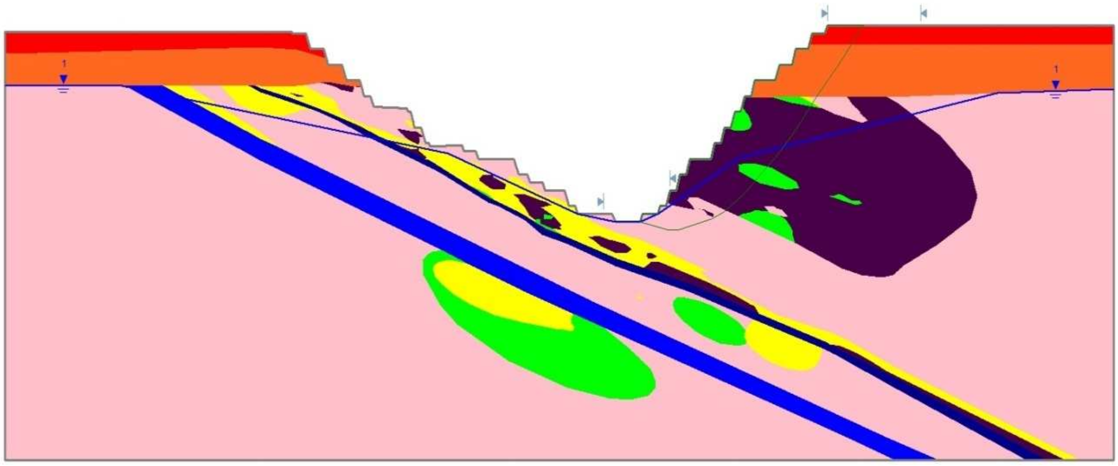


Figure 5-5: Illustration of Design Section NE with drained groundwater conditions for stability analysis

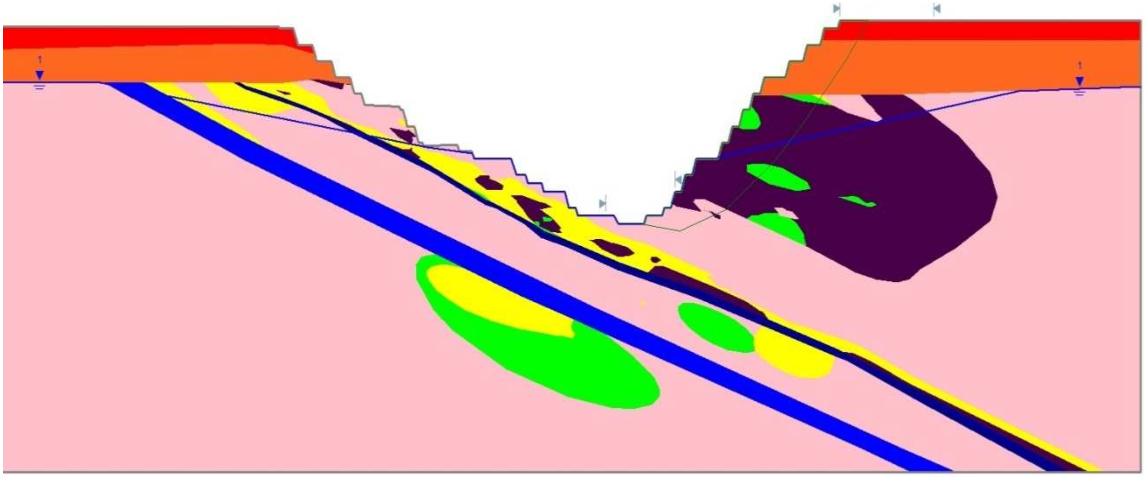


Figure 5-6: Illustration of Design Section NE with undrained groundwater conditions for stability analysis

5.8 Slope Geometry

The analysis was based on selected mine designs with conservative, best estimate and aggressive slope angles, to evaluate the sensitivity of stability to the overall angle. The 2008 feasibility study angles were considered a conservative slope design, due to the lack of structural information that was available at the time. Crest to toe overall angles through the un-weathered lithologies of 45° for the conservative, 50° for best estimate and 55° for the aggressive scenarios were used. The selected angles are presented as Table 5-4 and Figure 5-7. Ramp and berm positions on the sections were based on the mine plan *Otjikoto_mining_right_application_pit_design_v6_plot.dwg* developed by VBKom Consulting Engineers (Pty) Ltd which used the conservative overall slope angles.

Figure 3-4 (Section 3) illustrates the plan view of the proposed Otjikoto Pit with lines indicating the sections that were used for the limit equilibrium and the stress deformation analyses.

Table 5-4: Slope angles used for analysis

Section	Overall Un-weathered Slope Angle (°)*		
	Conservative	Best Estimate	Aggressive
North East	43	48	53
North West	33	-‡	-‡
Mid East	46	51	56
Mid West	26	-‡	-‡
South East	44	49	54
South West	43	48	53

* Crest to toe angle

‡ North west and mid west slopes were not increased as they are defined by the orebody

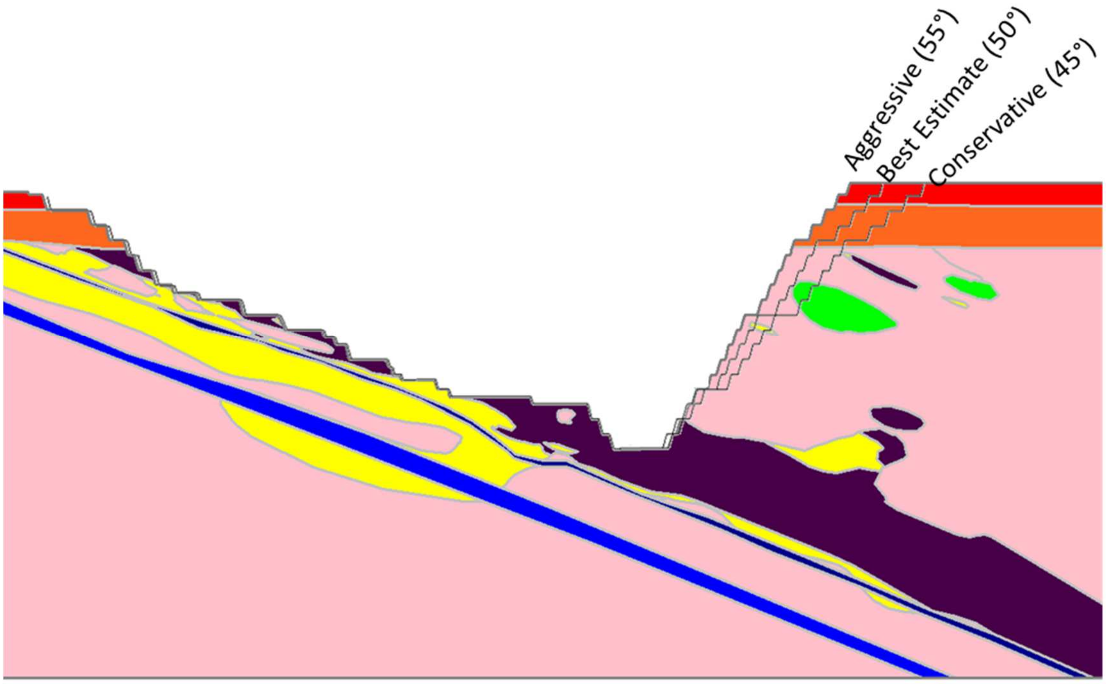


Figure 5-7: Schematic section representing the geometries used for the slope angle sensitivity analysis

6 Slope Stability Evaluation

The slope stability evaluation was carried out on the slopes for the east and west walls. The analysis of the overall stability was assessed by changing the stack angles of the slope and was carried out using the program SLIDE from Rocscience based on the limit equilibrium method. The overall slopes were also analysed using the stress deformation analysis program PHASE 2 which uses the shear strength reduction technique on a finite element model.

A series of models with explicit joints had been analysed using the finite element code PHASE2. For simplicity, only the two main joint sets, normal and parallel to bedding planes, were included. The joint set parameters are included as Table 6-1. The Shear Strength Reduction (SSR) technique in PHASE2 was used to calculate a Shear Reduction Factor (SRF) which is equivalent to a Factor of Safety (FoS).

Table 6-1: Average joint set parameters of the model

Geotechnical Unit Description	Joint Orientation	Joint Spacing (m)	Mohr-Coulomb	
			c (kPa)	ϕ (°)
Average for Model	21°	2.00	16	34
	89°	5.00	16	34

6.1 Overall Slope Stability

The overall stability of selected sections of the Otjikoto planned pit was analysed with both a limit equilibrium and finite element analysis. The analysis considered a best estimate wet condition (drained slope) and an undrained condition with the results shown in Figure 6-1 and Figure 6-2 for SLIDE and in Figure 6-3 and Figure 6-4 for PHASE2 respectively. Tables of results from the conservative, optimum and aggressive limit equilibrium and finite element analyses for the Otjikoto proposed pit are shown in Table 6-2 to Table 6-7. The full Slide and PHASE2 results are attached as Appendix C.

It is not considered at this stage that planned dewatering of the slope will be required.

The Phase2 analysis of the South West section which included a joint network representing the bedding is considered conservative as the dip direction of the bedding is oblique to the dip direction of the slope in this area, precluding large scale planar failure. Analysis of the slope excluding joint networks results in very high factors of safety as indicated in Table 6-5. As this is a three-dimensional problem the real factor of safety will between these end member states.

Although the stability of the bullnose in the centre of the pit is generally a concern and required a flatter slope angle, it is aided by the large berm present due to the geometry of the orebody and therefore does not require further consideration.

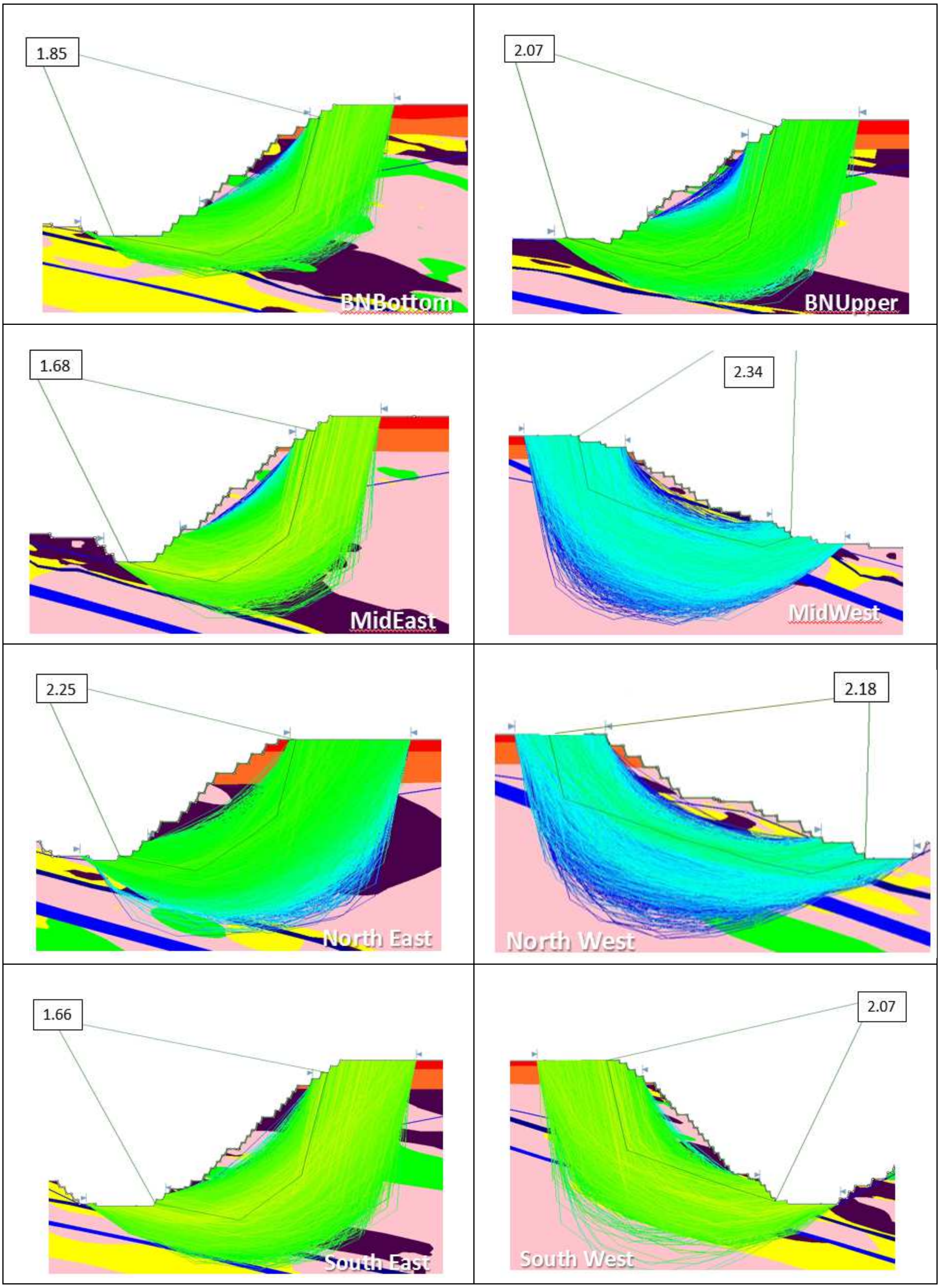


Figure 6-1: Stability of the drained overall slope in SLIDE

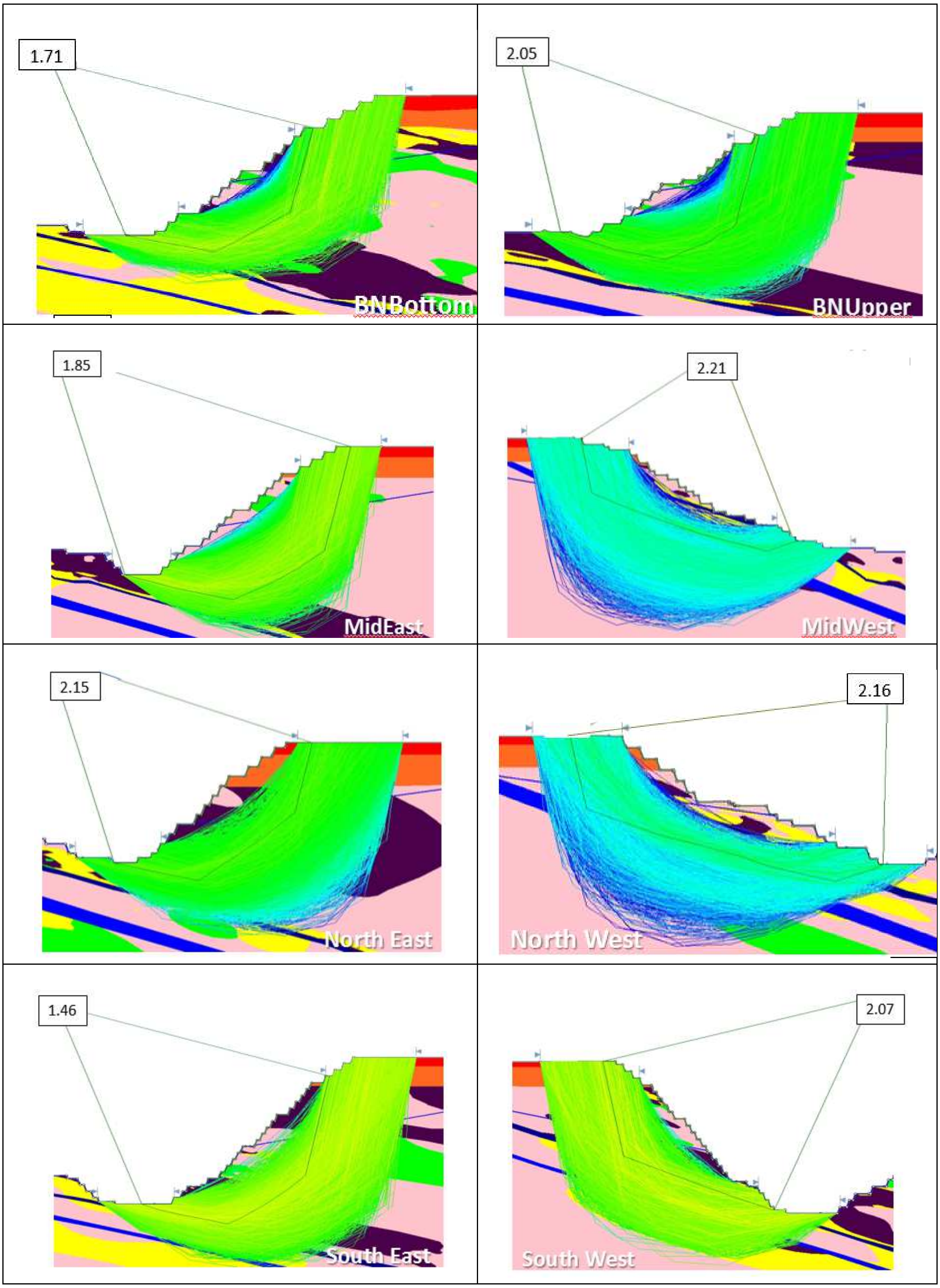


Figure 6-2: Stability of the undrained overall slope in SLIDE

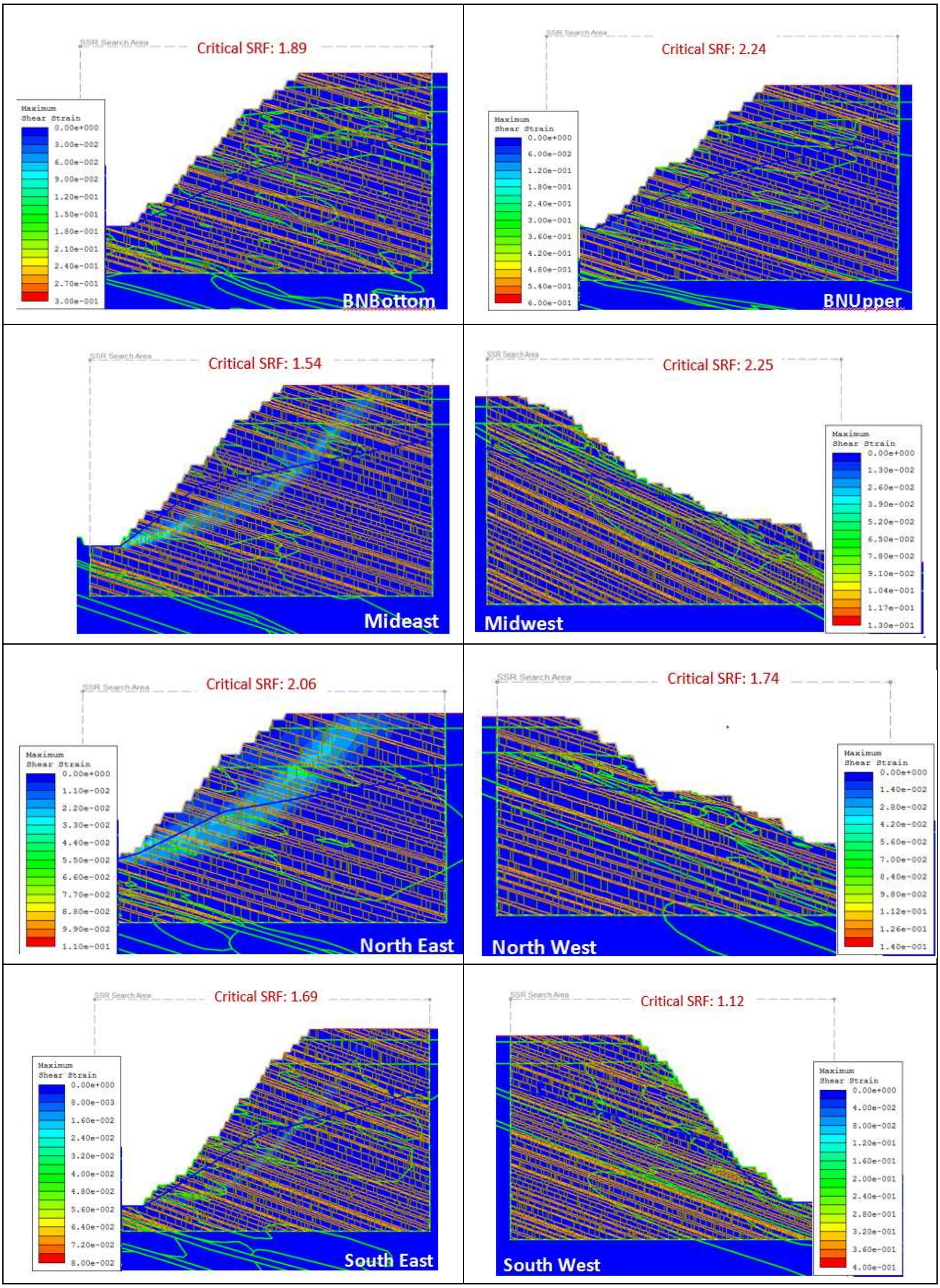


Figure 6-3: Stability of the drained overall slope in PHASE2

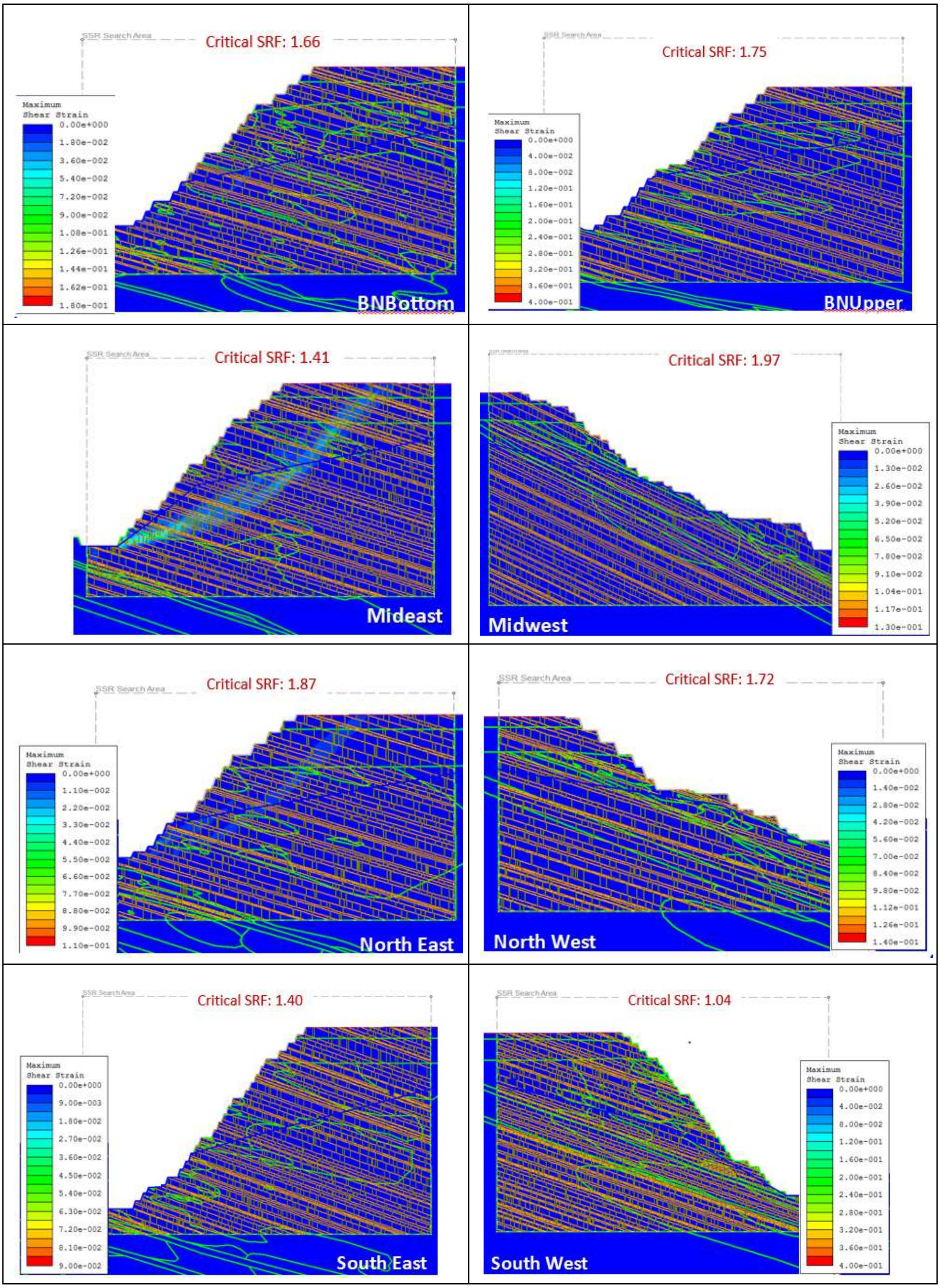


Figure 6-4: Stability of the undrained overall slope in PHASE2

Table 6-2: Results of SLIDE and PHASE2 analyses for the conservative models

Conservative							
Analysis Name	**Slope Angle (Toe-Crest)**	**SSR Area**	**Water Table**	**Joints**	**PHASE2 FOS**	**SLIDE FOS**	**SLIDE POF (%)**
BNBottom	45	East Wall	Drained	Cross	2.01	2.00	<1
BNUpper	41	East Wall	Drained	Cross	2.31	2.28	<1
Mideast	46	East Wall	Drained	Cross	1.68	1.80	<1
Midwest	26	West Wall	Drained	Cross	2.25	2.34	<1
Section NE	43	East Wall	Drained	Cross	2.17	2.21	<1
Section NW	33	West Wall	Drained	Cross	1.74	2.18	<1
Section SE	44	East Wall	Drained	Cross	1.75	1.78	<1
Section SW	43	West Wall	Drained	Cross	1.40	2.07	<1
Analysis Name	**Slope Angle (Toe-Crest)**	**SSR Area**	**Water Table**	**Joints**	**PHASE2 FOS**	**SLIDE FOS**	**SLIDE POF (%)**
BNBottom	45	East Wall	Undrained	Cross	1.57	1.82	<1
BNUpper	41	East Wall	Undrained	Cross	2.09	2.12	<1
Mideast	46	East Wall	Undrained	Cross	1.51	1.30	<1
Midwest	26	West Wall	Undrained	Cross	1.97	2.22	<1
Section NE	43	East Wall	Undrained	Cross	2.02	2.09	<1
Section NW	33	West Wall	Undrained	Cross	1.72	2.16	<1
Section SE	44	East Wall	Undrained	Cross	1.42	1.67	<1
Section SW	43	West Wall	Undrained	Cross	1.00	2.07	<1

Table 6-3: Results of SLIDE and PHASE2 analyses for the optimum models

Optimum							
Analysis Name	**Slope Angle (Toe-Crest)**	**SSR Area**	**Water Table**	**Joints**	**PHASE2 FOS**	**SLIDE FOS**	**SLIDE POF (%)**
50deg BNBottom	50	East Wall	Drained	Cross	1.89	1.88	<1
50deg BNUpper	46	East Wall	Drained	Cross	2.24	2.11	<1
50deg Mideast	51	East Wall	Drained	Cross	1.54	1.71	<1
50deg Section NE	48	East Wall	Drained	Cross	2.06	2.04	<1
50deg Section SE	49	East Wall	Drained	Cross	1.69	1.66	<1
50deg Section SW	47	West Wall	Drained	Cross	1.12	1.73	<1
Analysis Name	**Slope Angle (Toe-Crest)**	**SSR Area**	**Water Table**	**Joints**	**PHASE2 FOS**	**SLIDE FOS**	**SLIDE POF (%)**
50deg BNBottom	50	East Wall	Undrained	Cross	1.66	1.63	<1
50deg BNUpper	46	East Wall	Undrained	Cross	1.75	1.93	<1
50deg Mideast	51	East Wall	Undrained	Cross	1.41	1.60	<1
50deg Section NE	48	East Wall	Undrained	Cross	1.87	1.84	<1
50deg Section SE	49	East Wall	Undrained	Cross	1.40	1.51	<1
50deg Section SW	47	West Wall	Undrained	Cross	1.04	1.54	<1

Table 6-4: Results of SLIDE and PHASE2 analyses for the aggressive models

Aggressive							
Analysis Name	**Slope Angle (Toe-Crest)**	**SSR Area**	**Water Table**	**Joints**	**PHASE2 FOS**	**SLIDE FOS**	**SLIDE POF (%)**
55deg BNBottom	55	East Wall	Drained	Cross	1.75	1.85	<1
55deg BNUpper	51	East Wall	Drained	Cross	2.13	2.00	<1
55deg Mideast	56	East Wall	Drained	Cross	1.50	1.56	<1
55deg Section NE	53	East Wall	Drained	Cross	1.92	1.87	<1
55deg Section SE	54	East Wall	Drained	Cross	1.57	1.56	<1
55deg Section SW	53	West Wall	Drained	Cross	1.08	1.04	24
Analysis Name	**Slope Angle (Toe-Crest)**	**SSR Area**	**Water Table**	**Joints**	**PHASE2 FOS**	**SLIDE FOS**	**SLIDE POF (%)**
55deg BNBottom	55	East Wall	Undrained	Cross	1.49	1.53	<1
55deg BNUpper	51	East Wall	Undrained	Cross	1.85	1.79	<1
55deg Mideast	56	East Wall	Undrained	Cross	1.27	1.40	<1
55deg Section NE	53	East Wall	Undrained	Cross	1.22	1.61	<1
55deg Section SE	54	East Wall	Undrained	Cross	1.16	1.32	<1
55deg Section SW	53	West Wall	Undrained	Cross	1.01	0.94	86

Table 6-5: Results of PHASE2 analyses for the South West Section with no joint network

Analysis Name	Slope Angle (Toe-Crest)	SSR Area	Water Table	Joints	PHASE2 FOS
Conservative					
Section SW	43	West Wall	Drained	None	2.10
Section SW	43	West Wall	Undrained	None	1.90
Optimum					
Section SW	47	West Wall	Drained	None	1.93
Section SW	47	West Wall	Undrained	None	1.84
Aggressive					
Section SW	53	West Wall	Drained	None	1.62
Section SW	53	West Wall	Undrained	None	1.60

6.2 Sensitivity of Rock Mass Parameters

The sensitivity of UCS and GSI were modelled by increasing and decreasing the parameters to review the influence these parameters have to the slope stability analyses. Similar investigations have been carried out with regard to investigating the influence of various input factors (e.g. technical factors, material property factors and boundary condition factors) in numerical analyses and limit equilibrium analyses methods. Authors include Rathod and Rao (2012), Ureel and Momayez (2014) and Kanda and Stacey (2016).

The sensitivity analyses were focused on the limit equilibrium approach utilising the software programme SLIDE. The UCS input data was increased by using the average determined value per rock type and adding the standard deviation (Upper UCS). Conversely the value was decreased by subtracting the standard deviation from the average value (Lower UCS). The input parameters are tabled in Chapter 4, Table 4-2. GSI was increased (Upper GSI) and decreased (Lower GSI) more simplistically by simply adding or subtracting 10 respectively from the original determined value. Analyses excluded the western sections, as these slope angles were determined by the dip of the ore body.

The results of the sensitivity analysis in presented in Table 6-6. It can be understood by the results that UCS and GSI play a significant role in the determination of FOS. GSI has a much larger influence on the FOS than the UCS value. The use of the average rock mass property values (e.g. UCS and GSI) for the analyses is deemed suitable so as to not support the determination of unrealistically high (conservative design) or low (aggressive design) FOS values.

Table 6-6: Results of SLIDE Sensitivity Analysis

Analysis	Original Results	Lower UCS	Upper UCS	Lower GSI	Upper GSI
Conservative Drained					
BNBottom	2.00	1.71	2.27	1.58	2.62
BNUpper	2.28	1.96	2.54	1.73	3.04
Mideast	1.80	1.52	2.02	1.40	2.31
Section NE	2.21	1.88	2.50	1.71	2.90
Section SE	1.78	1.52	1.98	1.38	2.31
Conservative Undrained					
BNBottom	1.82	1.53	2.06	1.41	2.40
BNUpper	2.21	1.80	2.38	1.60	2.89
Mideast	1.73	1.46	1.95	1.33	2.26
Section NE	2.09	1.75	2.36	1.60	2.77
Section SE	1.67	1.43	1.87	1.29	2.20
Optimum Drained					
BNBottom	1.88	1.60	2.11	1.48	2.42
BNUpper	2.11	1.82	2.37	1.62	2.86
Mideast	1.71	1.45	1.91	1.31	2.26
Section NE	2.04	1.72	2.30	1.57	2.66
Section SE	1.66	1.41	1.86	1.29	2.14
Optimum Undrained					
BNBottom	1.63	1.36	1.86	1.25	2.17
BNUpper	1.93	1.63	2.18	1.43	2.65
Mideast	1.60	1.34	1.81	1.19	2.14
Section NE	1.84	1.53	2.12	1.45	2.51
Section SE	1.51	1.26	1.70	1.16	1.99
Aggressive Drained					
BNBottom	1.9	1.6	2.1	1.4	2.4
BNUpper	2.0	1.7	2.2	1.5	2.7
Mideast	1.6	1.3	1.8	1.2	2.1
Section NE	1.9	1.6	2.1	1.4	2.4
Section SE	1.6	1.3	1.8	1.2	2.0
Aggressive Undrained					
BNBottom	1.53	1.26	1.78	1.14	2.05
BNUpper	1.79	1.49	2.05	1.30	2.57
Mideast	1.40	1.16	1.60	0.96	1.99
Section NE	1.61	1.34	1.85	1.23	2.20
Section SE	1.32	1.13	1.52	1.01	1.74

6.3 Rock Fall Analysis

The results from the overall slope and stack stability analyses showed that the overall factor of safety was higher than 1.5.

It was identified that a more significant risk to the operation is that caused by rock fall material, which ultimately may report beyond the geotechnical berm and injure persons working in the pit and/or damage equipment that is operating in the pit.

An intensive rock fall analysis was conducted for the proposed Otjikoto pit. The analyses considered the stacks and the potential impact of various designs:

– Single benching (Figure 6-5)
– Double benching (Figure 6-6)
– Overall stack angles on rock fall incidents

The rock fall analyses were conducted using the rock fall incidents Rocscience software package *RocFall 4.0.* Combinations of 50 m high stacks with 10 m and 20 m high benches and crest to toe slope angles of 50° and 55° were considered with respect to a 15 m wide geotechnical berm, to model the worst case scenario in terms of rock falls. From these models, probabilities of rock fall extending beyond the geotechnical berm had been determined. Benches were assumed to have crest damage of 2 – 3 m, along with 2 m frozen toes, as is common in many open pit operations. When assuming the material's ability to arrest falling rocks, berms were assumed to have "talus cover" which equates to blasted material rather than bare rock, and bench face properties were assumed to be of "clean hard rock".

The rock fall analyses showed that the probability of rocks bouncing beyond the geotechnical berm are the highest where smaller bench heights were chosen with steeper stack angles. The RocFall analysis also shows the benefit of a double bench configuration (i.e. 20 m benches) for the stacks. Figure 6-7 illustrates an example of the rock fall analysis and Table 6-7 show the results of the rock fall analysis.

Cognisance was taken of the fact that not all the material mobilised during a rock fall will report beyond the geotechnical berm. Given the geometry of the pit and the characteristics of the rock mass, it is considered that no one portion of the pit is more vulnerable to a rock fall than any other portion.

The probability of material becoming mobilised was assessed as a function of size and frequency. There was a higher probability of smaller rocks becoming mobilised and the frequency of rocks reporting beyond the ramp was considered to be infrequent.

Owing to the fact that the slope design feasibility study recommended a 15 m geotechnical berm, the results for the probabilities of rocks falling beyond the 15 m geotechnical berm were carried forward to the next step of investigation – i.e. the event tree.

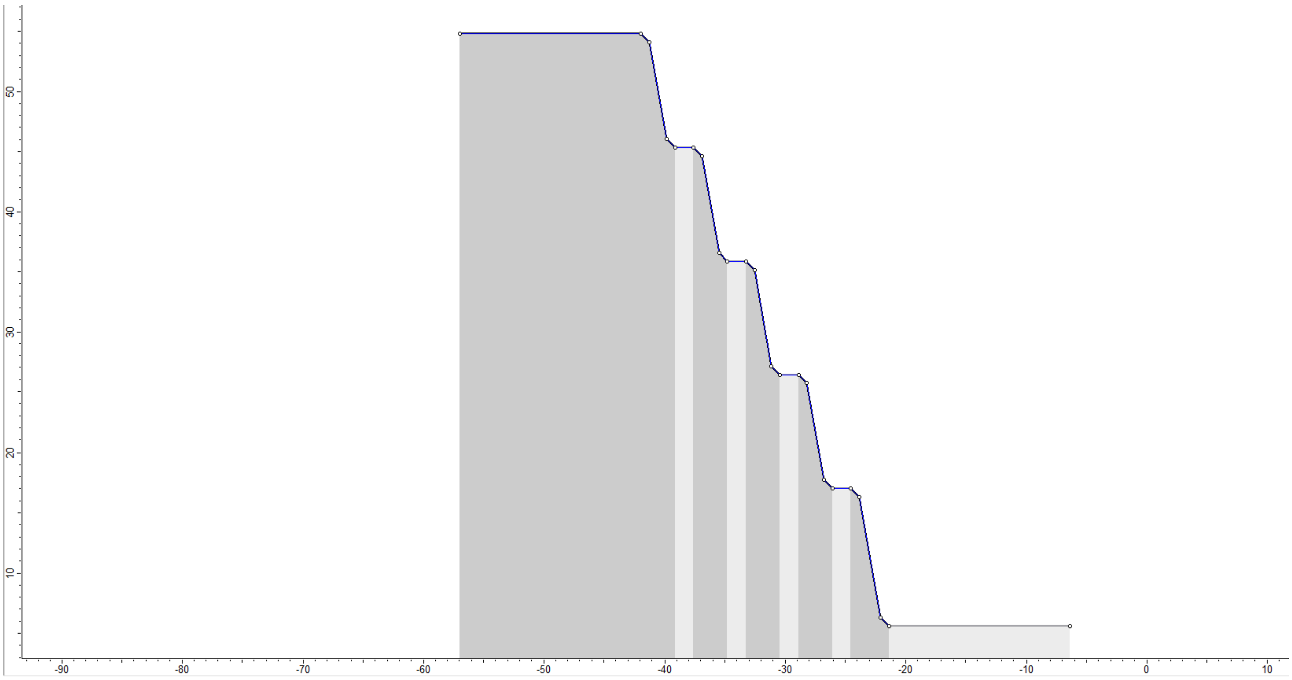


Figure 6-5: Example of RocFall geometry with single benching option

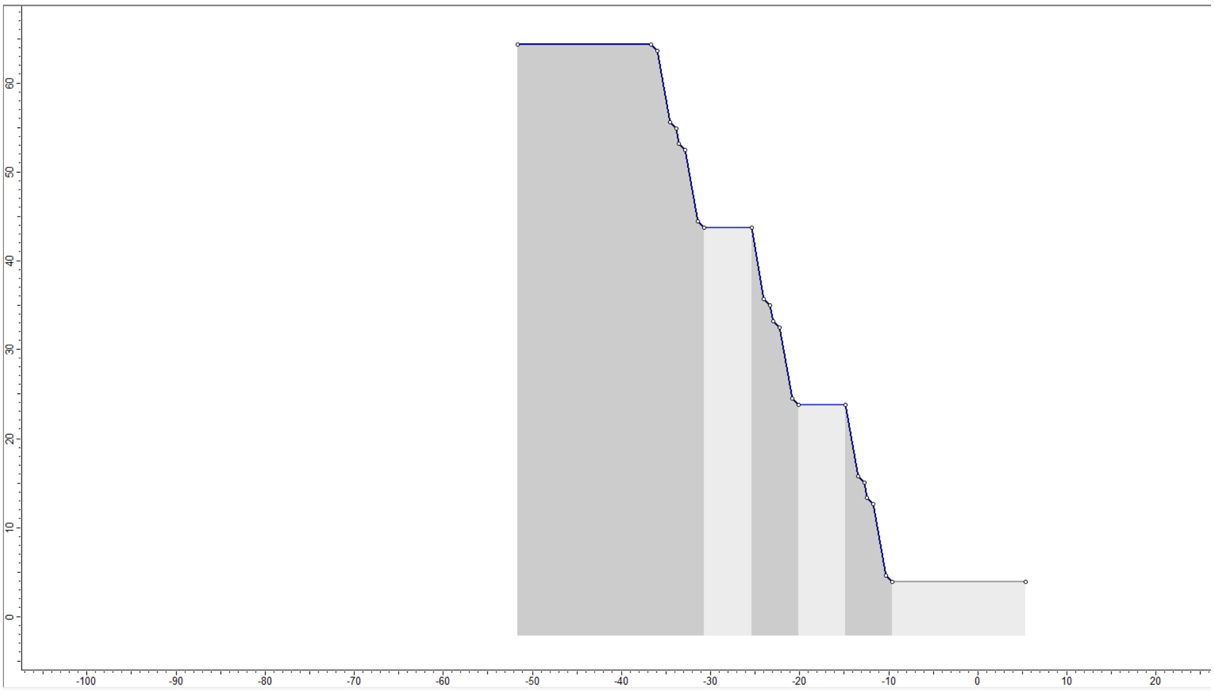


Figure 6-6: Example of RocFall geometry with double benching option

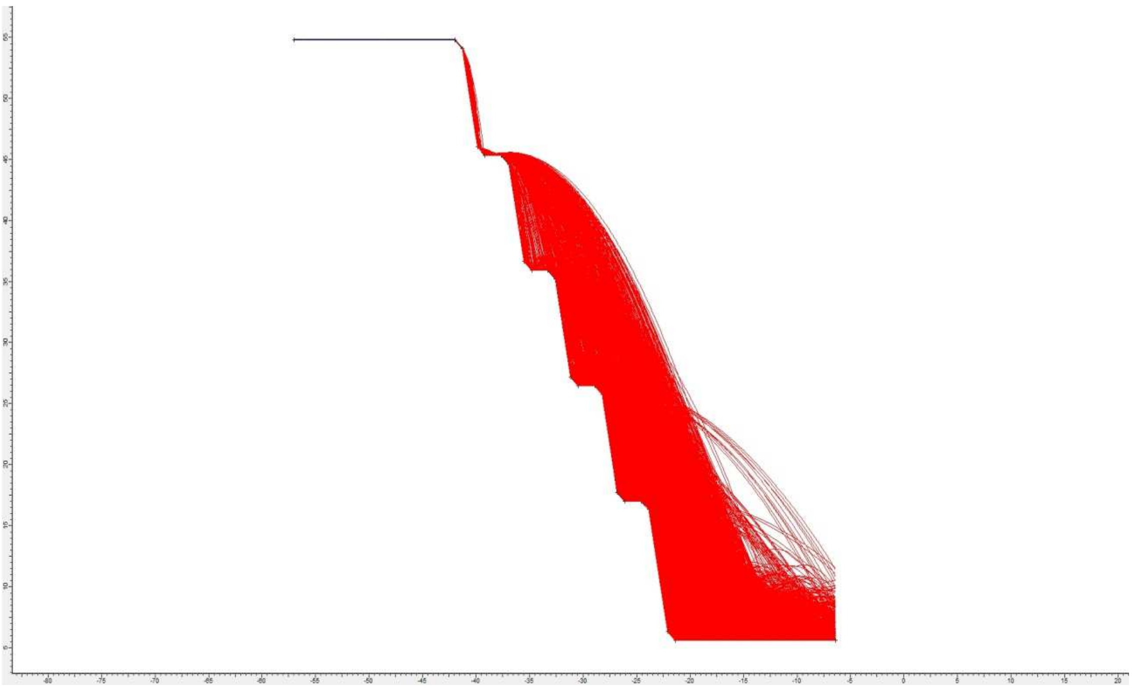


Figure 6-7: Example of RocFall results (showing results for 65° stack, 50m stack with 10m benches)

Table 6-7: Probability of rocks reporting beyond a 15 m geotechnical berm

RocFall (5000 Rocks)	Single Benching	SE50	SW50	SE55	SW55
Number of Rocks Passing 15m		863	687	1264	1098
Percentage Passing		17%	14%	25%	22%
RocFall (5000 Rocks)	**Double Benching**	**SE50**	**SW50**	**SE55**	**SW55**
Number of Rocks Passing 15m		504	567	1059	699
Percentage Passing		10%	11%	21%	14%

6.4 Consequence Analysis of Rock Falls

6.4.1 Exposure Analysis of Personnel and Equipment

The probability of coincidence in time and space of mine personnel with an eventual rock fall event was estimated for:

- Scenarios considering the extent of rock fall incidents
- Scenarios considering the efficacy of visual detection and instrumentation detection
- Scenarios considering the efficacy of evacuations of personnel and equipment out of the pit
- Scenarios considering the extent of exposure of personnel and equipment

The approach uses information on personnel and the equipment fleet working during the year to assess the percentage of time in a year that workers are exposed to rock fall events.

The data regarding total personnel required as well as equipment required to operate the pit were obtained from VBKom Consulting Engineers (Pty) Ltd (*Rockfall Risk Analysis data, Sheet: Labour_Owner.xlsx)*. Similarly, cycle times were also obtained (*Rockfall Risk Analysis data, Sheet: Cycle Times Talpac.xlsx)*.

Table 6-8 shows the derivation of available hours per year using modifying factors from the data (After, VBKom 2012).

Table 6-8: Available operating hours

(A) Available Calendar Time	
Calendar Time (days)	365.25
Sub-total (days)	**365.25**
(B) Time not available per year	
Annual Closure	5
Weather Interruptions	7
Blast delays (days)	5.42
Shift delays (days)	61.45
Sub-total (days)	**78.87**
(C) Other considerations	
Mechanical availability (%)	81%
Available hours per year	
(D) hours = (A – B) x (C) x 8 x 3	**5567**

The data extracted from VBKom shows that the number of operational personnel required increases year on year, from 189 in 2013 to 271 persons in 2019 and then decreases to 19 in 2027. This is shown in Figure 6-8. Similarly, the production and support equipment increases from 55 in 2013 to 92 in 2019 and decreases to 7 in 2027. This is shown in Figure 6-9.

As a result, the exposure of mine personnel and equipment in the pit will increase over time and the analyses conducted therefore considered both the interim exposures as well as the exposure in later years.

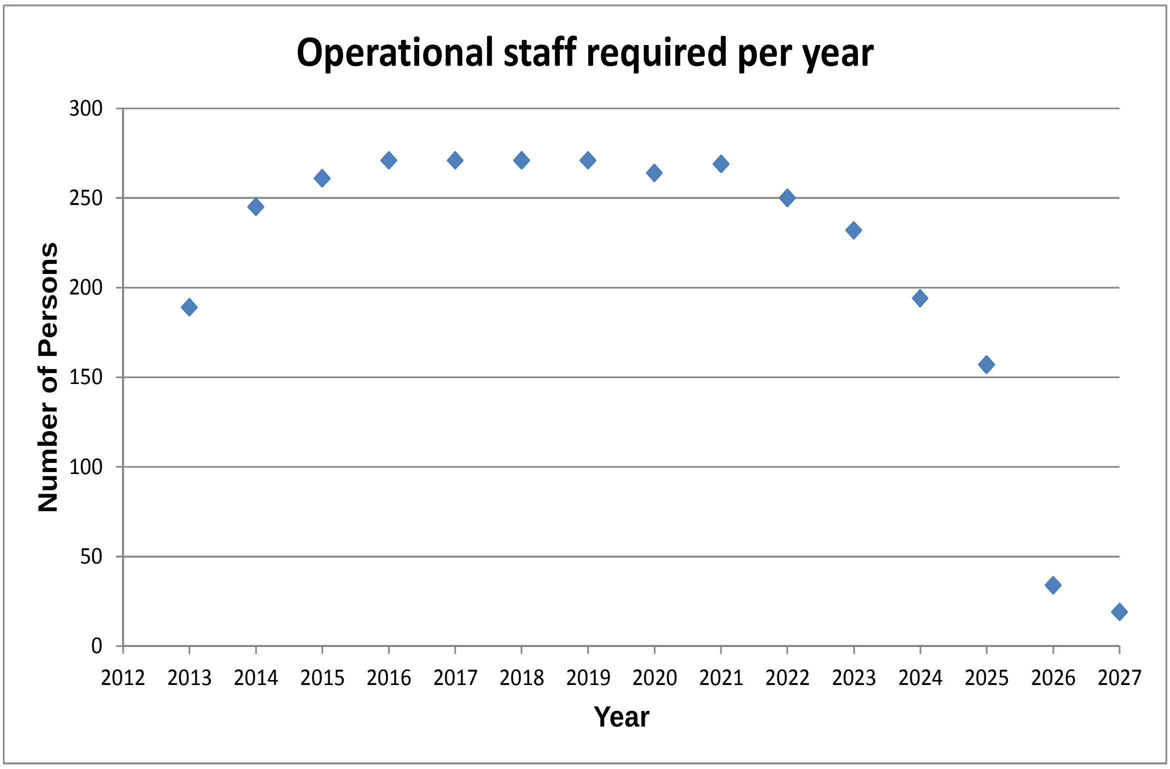


Figure 6-8: Number of operational personnel required for the mine (Modified from VBKom, 2012)

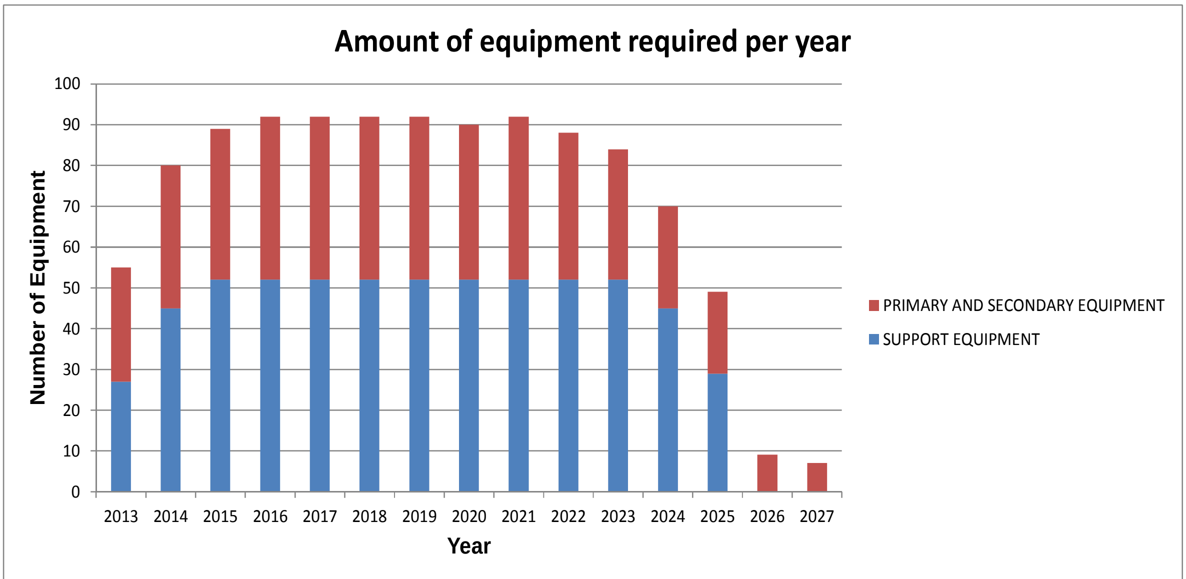


Figure 6-9: Equipment required for the mine (Modified from VBKom, 2012)

Raw exposure values represent the percentage of time in a year spent by the different workers within the mine. However, these values need to be adjusted to account for the fact that the likelihood of impact from an event is also determined by the location of the worker relative to the slope as well as by the mobility of the equipment dictated by the mining operation routines.

An "in pit factor" was assigned to relate to the percentage of time that the equipment units (and therefore operators) spend within the pit, an "exposure factor 1" was assigned to relate the percentage of time within the more active zones of the pit and an "exposure factor 2" was assigned to relate the percentage of time within the most critical zones of the pit. The exposure values of personnel and equipment are shown in Table 6-9.

Table 6-9: Exposure analysis of personnel and equipment

PRIMARY AND SECONDARY EQUIPMENT FLEET SIZE	Number of Units	Number of operators	Operating hours required per unit / year	Total hours per year	Raw exposure	In pit factor	Exposure factor 1	Exposure factor 2	Critical exposure
Haul Trucks	15	15	5 516	5567	99%	0.65	0.33	0.12	3%
Small Shovels	1	1	5 292	5567	95%	1.00	0.75	0.12	9%
Large Shovels	2	2	5 292	5567	95%	1.00	0.75	0.12	9%
Drill Rigs	7	7	5 182	5567	93%	1.00	0.50	0.12	6%
Front End Loaders	2	2	5 051	5567	91%	1.00	0.75	0.12	8%
Small Track Dozers	2	2	5 067	5567	91%	0.90	0.75	0.12	7%
Large Track Dozers	2	2	5 067	5567	91%	0.10	0.75	0.12	1%
Graders	3	3	4 105	5567	74%	0.50	0.30	0.12	1%
Water Trucks	2	2	3 339	5567	60%	0.50	0.30	0.12	1%
Diesel Bozers	2	2	3 339	5567	60%	0.50	0.50	0.12	2%
Wheel Dozers	2	2	4 589	5567	82%	0.90	0.30	0.12	3%
SUPPORT EQUIPMENT									
FLEET SIZE									
Mobile Lube truck	2	2	1 530	5567	27%	0.50	0.50	0.12	1%
Mobile Service truck	2	2	1 530	5567	27%	0.50	0.50	0.12	1%
Tyre Handler	1	1	500	5567	9%	0.00	0.00	0.12	0%
Roller / Compactor	1	1	1 392	5567	25%	0.50	0.50	0.12	1%
Rock Breaker	2	2	3 000	5567	54%	0.90	0.75	0.12	4%
Lightning Plant	10	10	2 320	5567	42%	1.00	0.00	0.12	0%
Crane	1	1	500	5567	9%	0.05	0.00	0.12	0%
Skid Steer Loader	2	2	2 088	5567	38%	0.50	0.30	0.12	1%
Hi Lift Crane / Access Platform	1	1	1 044	5567	19%	0.00	0.00	0.12	0%
Forklift	1	1	1 044	5567	19%	0.00	0.00	0.12	0%
Light Delivery Vehicle	23	23	1 500	5567	27%	0.50	0.30	0.12	0%
Busses	4	4	1 500	5567	27%	0.50	0.00	0.12	0%
Lowbed and truck	1	1	500	5567	9%	0.50	0.50	0.12	0%
Trenches Loader Backhoe	1	1	500	5567	9%	0.50	0.30	0.12	0%

Table 6-9 indicates that the critical equipment units, in terms of exposure, are the shovels and the front end loaders. Based on pit volume and the number of shovels, a critical area calculation was done based on the percentage of critical area the shovels will be in, in terms of the volume of the proposed pit. This value equates to 2 %.

The critical exposure calculated was carried forward to the event tree analysis.

6.4.2 Event Tree Analysis for Safety Impact

Event trees were used to estimate the probability of fatality and injury (with regard to mine personnel) caused by the occurrence of rock fall events. Event trees were analysed for overall slope angles that employed a ramp width of 15 m (based on recommendations from the feasibility study).

The event tree is a diagram that connects the starting event (rock fall) with the ultimate consequence under evaluation (fatality or injury) through a series of intermediate events based on a cause-effect relation. The events are quantified in terms of their likelihood of occurrence, thus enabling the assessment of the end outcomes in terms of their probabilities of occurrence, following the appropriate rules to operate the AND/OR gates.

The typical structure of the event tree used for the analysis of safety impact of rock fall events is shown in Figure 6-10 and demonstrates that the components of the tree address the following questions:

- Does visual inspection detect movement?
- Is there sufficient warning from visual alert?
- Does the monitoring system detect the event?
- Is there sufficient warning from the monitoring system?
- Is the evacuation procedure successful?
- Are there people exposed?
- Is there at least one fatality?
- Is there at least one injury?

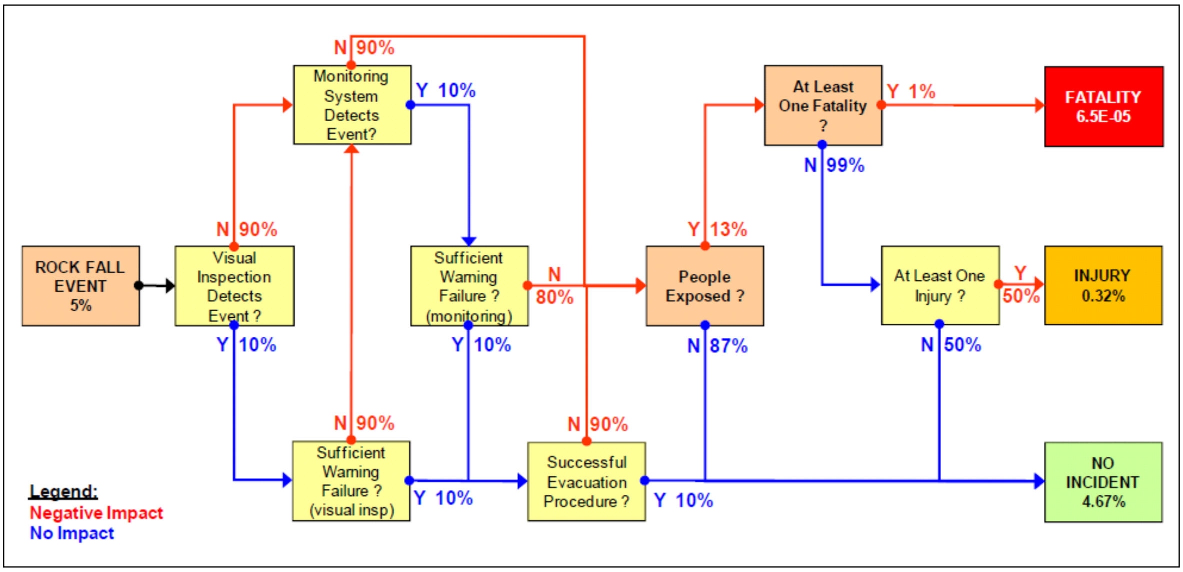


Figure 6-10: Example of an event tree for the analysis of safety impact of instability events

The probabilities assigned to the events in the tree are a reflection of aspects such as type and size of liberated material, proximity of equipment and personnel and similar types of considerations. The probability of a particular consequence in the tree is calculated as the sum of the probabilities of the various possible sequences of events leading to that consequence. For instance, a fatality can be the result of 7 possible sequences of events according to the diagram of Figure 6-10. The probability of each sequence is calculated as the product of the probabilities of the events in that sequence.

Due to the subjectivity implicit in the assessment of the probability values (P) assigned to the event tree components, it is important to account for the uncertainty in these judgmental probabilities.

This was carried out by using triangular distributions defined by upper and lower values for each entry to represent the uncertainty of the input components. Figure 6-11 illustrates the assumption of uncertainty in judgment of the probability (P) values used for the event tree (ET) calculations. It corresponds to a maximum variability of 20% which is reduced linearly as the P values get closer to the limits of the range of variation.

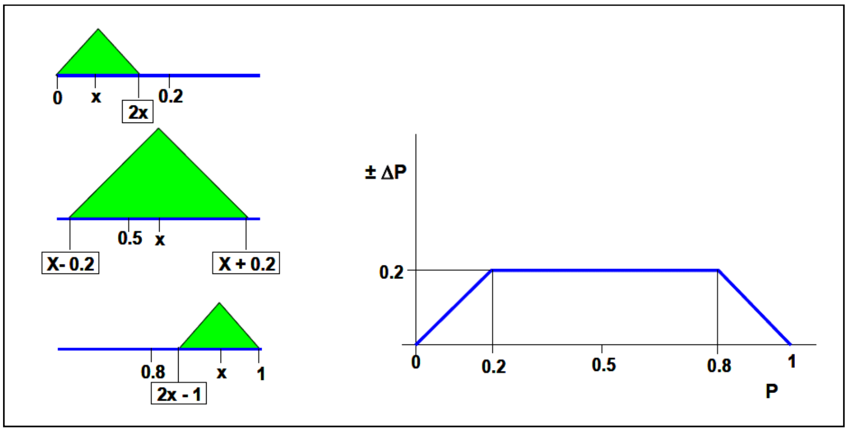


Figure 6-11: Assumption of uncertainty in the assessment of the inputs (P) to the event tree (Oracle Crystal Ball)

Using Oracle's *Crystal Ball* function, Monte Carlo analyses of the event trees are then carried out to enable the definition of distributions for each consequence calculated, thus enabling the attainment of confidence levels to the results.

Figure 6-12 shows the typical results of the Monte Carlo analysis of the ET presented in Figure 6-10 for safety impact from rock fall events.

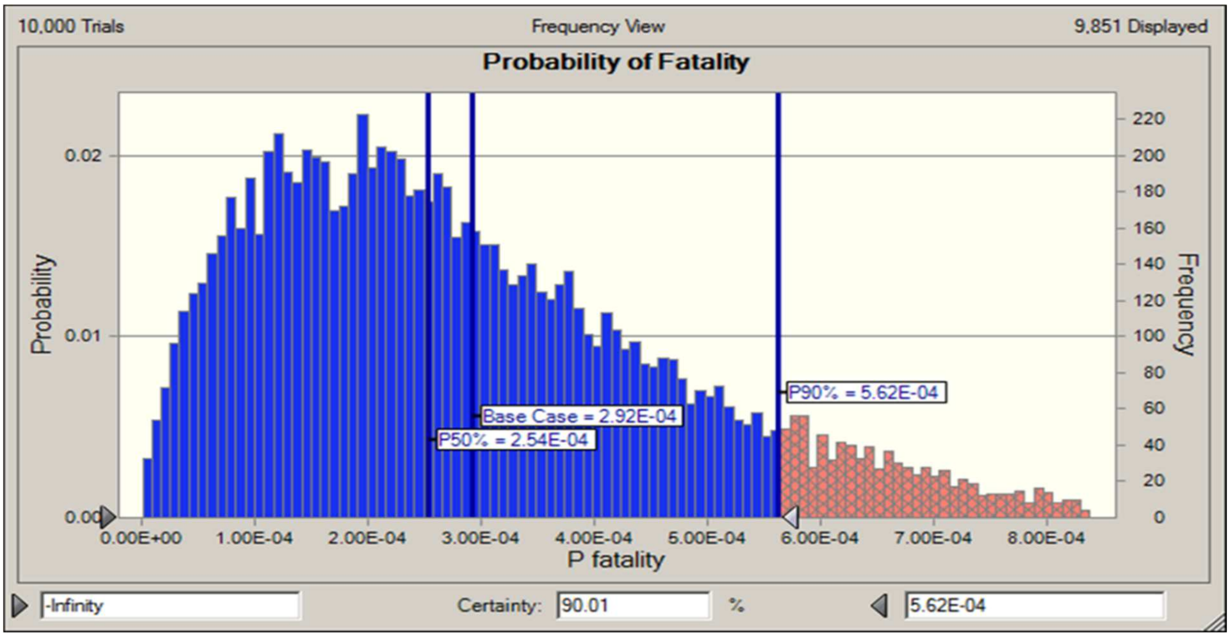


Figure 6-12: Example of typical results of a Monte Carlo analysis of the event tree showing frequency distribution (Oracle Crystal Ball)

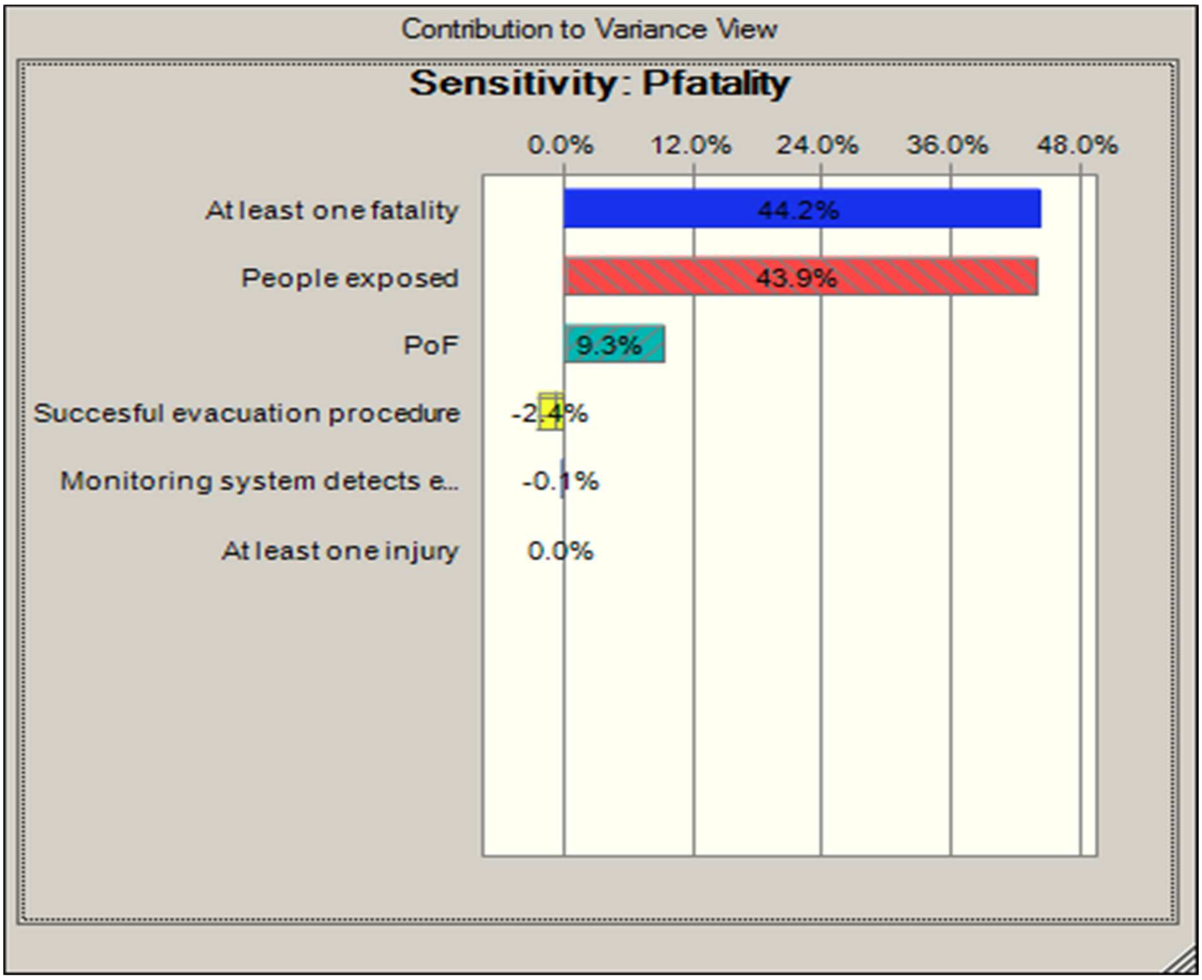


Figure 6-13: Example of typical results of a Monte Carlo analysis of the event tree showing tornado graph (Oracle Crystal Ball)

The frequency distribution at the left of the figure indicates that the base case P fatality value is $6.5x10^{-5}$ but a more conservative estimation corresponds to $1.3x10^{-4}$ which has a 90% confidence as determined by the relative position of this value within the whole distribution of possible values.

Another important result of the stochastic analysis of the ET is represented by the tornado graph at the right of Figure 6-12. This graph shows the relative contribution of the various inputs of the ET to the variance of the calculated probability of fatality, thus identifying those factors of major relevance for the analysis requiring a more detailed assessment. The tornado graph of Figure 6-12 shows that the issues of exposure of personnel and probability of occurrence of rock fall events are the more relevant aspects in the example.

6.4.3 Benchmark Criteria for Safety Impact

This chapter is based on the discussion presented in Section 2.2 of the literature review. The discussion is repeated to highlight the importance of the benchmark criteria as it is a significant consideration required for the final result output.

The comparison of the calculated P fatality values to available benchmark criteria is important to meaningfully interpret the results. A typical graph used for this purpose is indicated in Figure 6-14; which presents acceptability criteria for fatalities defined by different sources (Tapia et al. 2007). The areas in the graph related to tolerability of risk are derived from risk guidelines developed in various countries and correspond to risk thresholds in terms of local acceptability of deaths from industrial and other accidents (HSE, 2001). It is plotted as the number of fatalities from accidents versus the annual probability of exceeding that number.

The “Local Tolerability Line” defines a region which is characterized by both high frequencies and severe consequences (the “Intolerable” region). The region between this line and the “Local Scrutiny Line” is a region of possibly unjustifiable risk. Between this latter line and the “Negligibility Line” is a region which is judged to be tolerable but for which all reasonably practicable steps should be taken to reduce the hazard further. This is the ALARP region (As Low As Reasonably Practicable). All combinations of frequency and number of fatalities which fall below the “Negligibility Line” are considered to be negligible.

The graph also contains fatality criteria developed by the dam engineering discipline, which is of interest as bench mark criteria, because it is based on a conservative approach to risk, since the consequences of dam failures are so devastating. Of particular interest is the single fatality at an annual probability of 10-4 identified as “The proposed BC Hydro individual risk”. This number represents the risk exposure to ‘natural death’ by the safest population group in North America aged between 10 to 14 years within a person’s life cycle. The interpretation is that 1 in 10,000 persons between the ages of 10 and 14 years will die every year from natural causes (HSE, 2001). This is also popularly accepted as the boundary between voluntary and involuntary risk. Accepting this as a criterion for slope design, implies, therefore, that a person subjected to the open pit working environment is not exposed to greater death risk resulting from a slope failure than he is of dying due to natural causes (SRK, 2006).

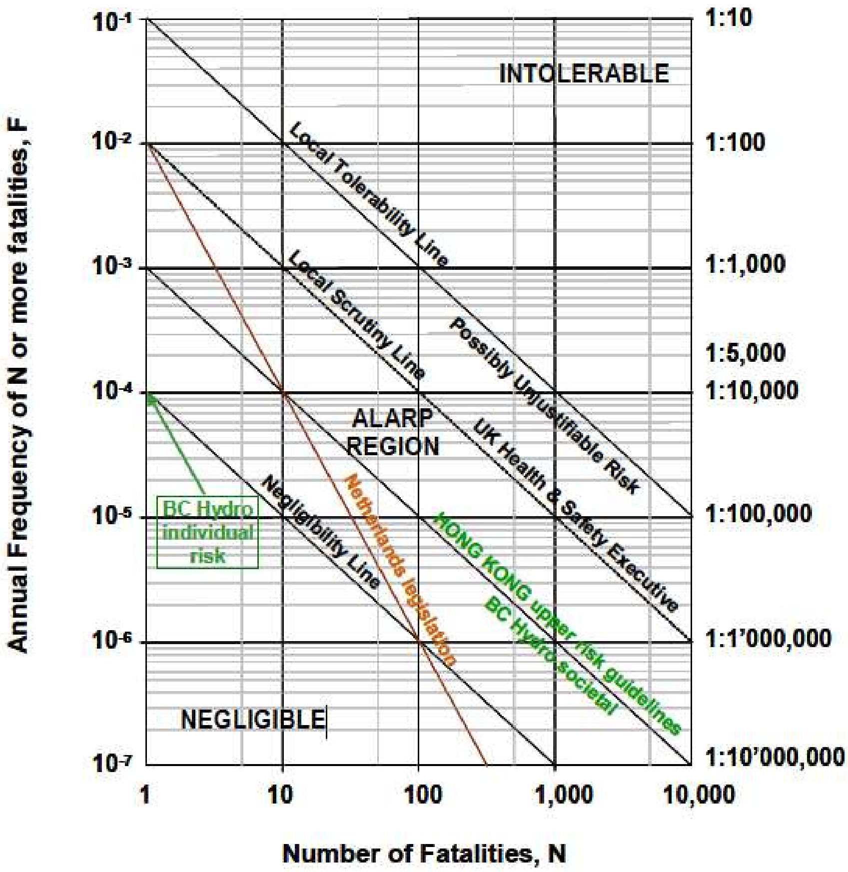


Figure 6-14: Benchmark criteria considering extreme eventualities per annum, used in engineering disciplines (where ALARP = As Low As Reasonably Practicable) (Steffen *et al.,* 2008)

6.4.4 Event Tree Analysis for Equipment Impact

The consequence analysis for the assessment of impact of slope failures on equipment is very similar to that described for personnel safety impact. The event tree for this analysis contain similar questions to those used for the personnel safety analysis, and the same arguments given to support the inputs are applicable, but in this case, the new values refer to impact on equipment rather than on workers. In general the values for equipment reflect the lesser mobility and more difficult successful evacuation of equipment rather than personnel. The typical event tree for the analysis of impact on equipment of slope instability events is shown in Figure 6-15. The relevant impacts selected for the assessment of risks on equipment from rock fall instability range from "equipment damaged" to "total loss of equipment".

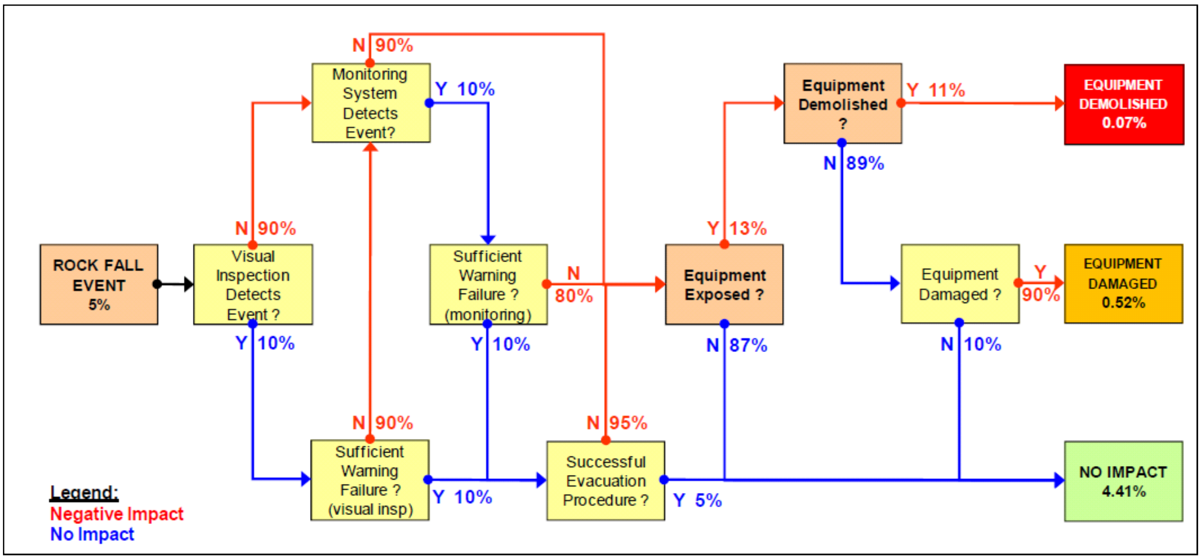


Figure 6-15: Example of an event tree for the analysis of equipment impact of instability events

The fault tree relevant to the proposed Otjikoto Pit, with no slope management programme in place, is presented as Figure 6-16, and with a slope management programme, presented as Figure 6-17. The full event tree analyses are attached as Appendix D.

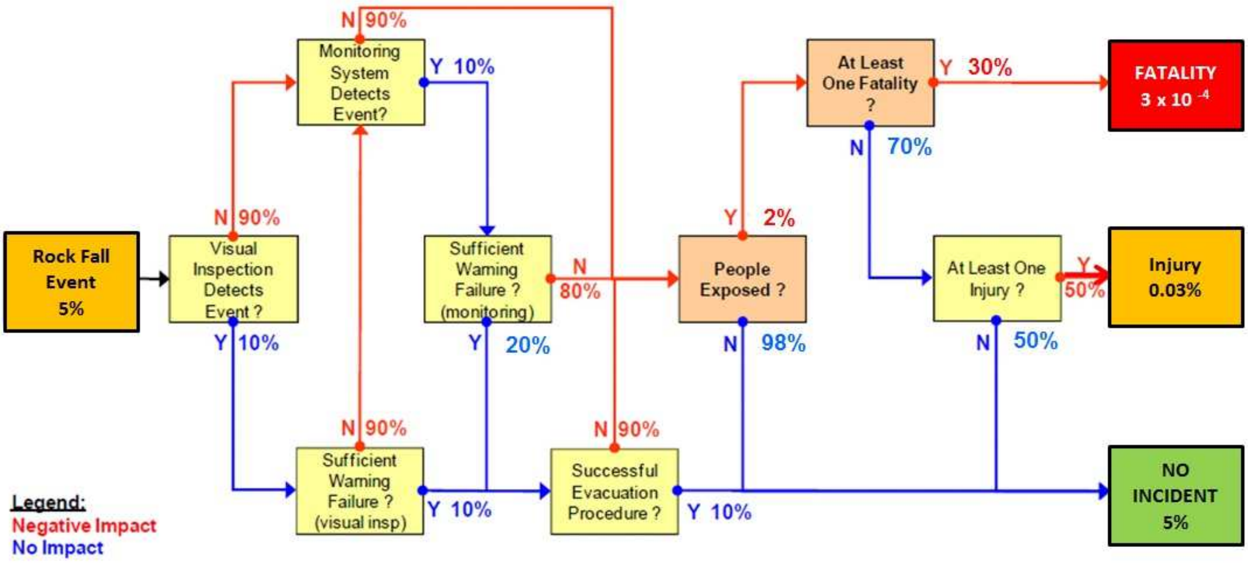


Figure 6-16: Safety event tree for a slope with no geotechnical systems in place

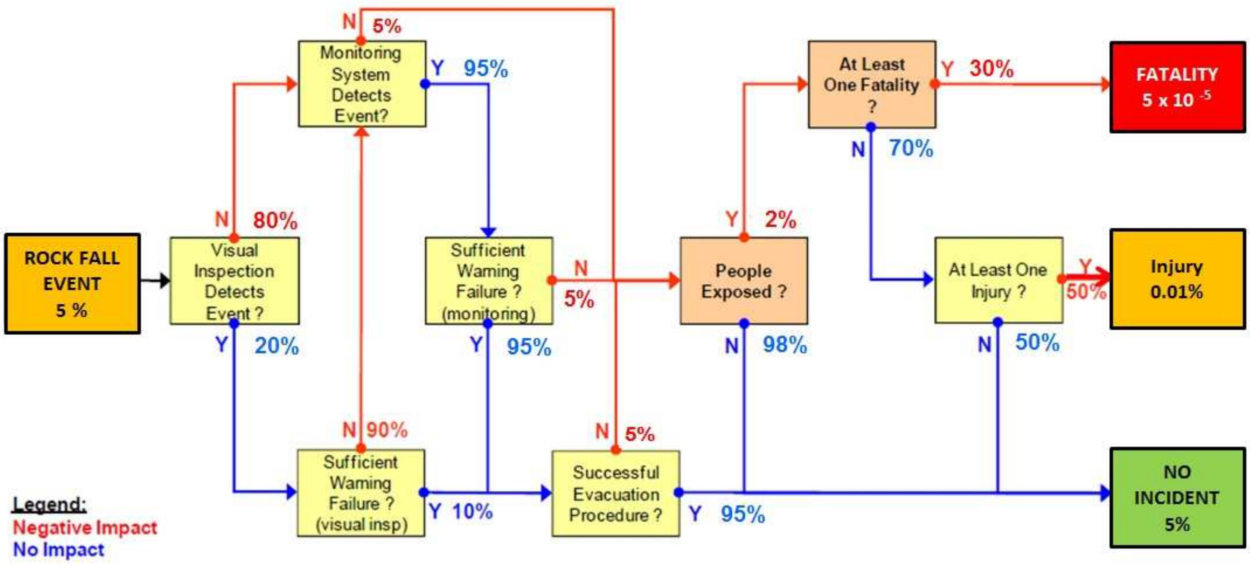


Figure 6-17: Safety event tree for a slope with geotechnical systems in place

Results of the fault tree analyses with various probabilities of failure from a rock fall event are presented as Figure 6-18, which indicates a recommended design probability of failure of 10% for the overall stability from a rock fall perspective.

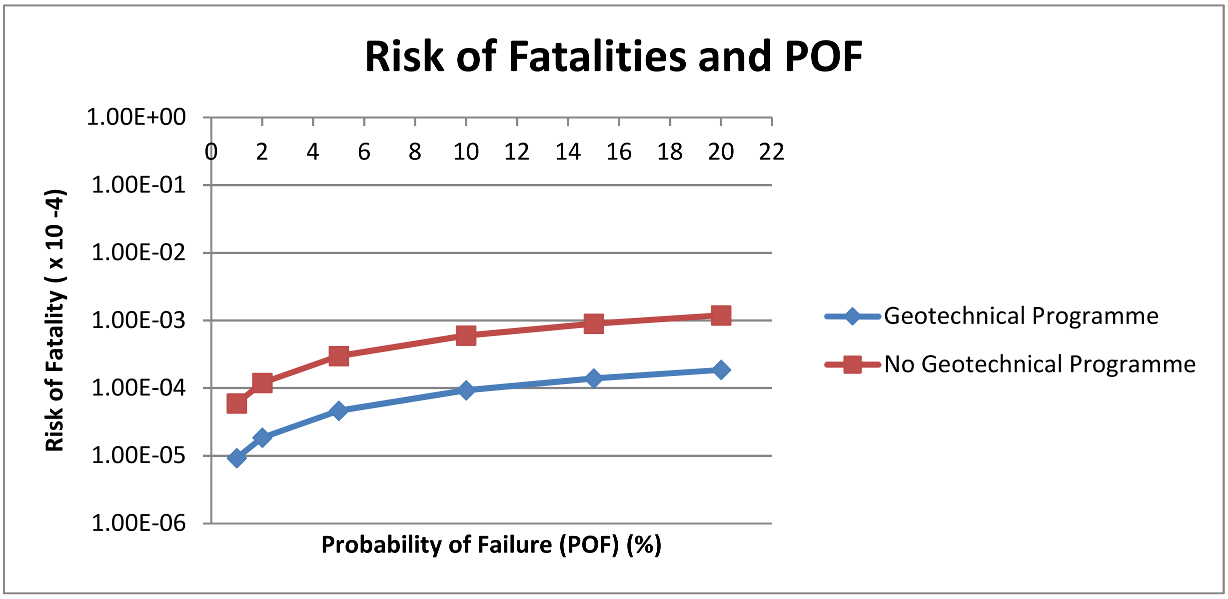


Figure 6-18: The effect of PoF on fatality risk

6.5 Identification of Mitigation Options

The identification of risk mitigation options for those cases where the calculated risks exceed the acceptability criteria is the final step of the risk evaluation process. Because the greatest risk to the operation is not strictly considered to be a slope failure but rather a problem of rock fall, the mitigation aspects will be aligned accordingly.

Risk mitigation options should be sought initially by reducing the consequences of rock falls. Primary options of risk mitigation are aimed at reducing the effects of incidents through slope management procedures.

In this case, the inspection of the event tree diagrams used for impact assessment enables the identification of slope management improvements to provide the maximum cost-benefit at the required acceptability criteria.

An overall stack angle of 65˚ with a ramp width of 15 m limits the quantity of rock fall material reporting past the ramp and ensures significant safety for personnel and equipment.

6.6 Pit Slope Management

A pit slope management programme is utilised to optimise slope geometry in respect of the rock mass quality and condition. Responsibility for the effective functioning of the slope management plan must be assigned to a person with the necessary authority (typically, the Geotechnical Section within the Mine Engineering Department) though support will be required from other mining disciplines (e.g. the Geology, Survey, Planning and the Operations Departments).

- Pit limit blasting programme
  - A pit limit blasting programme is a pre-requisite to minimise damage to the rock mass behind the mine design line and reduce the requirements for scaling on interim and final pit walls.
- Cleaning of interim and final pit faces
  - Potentially unstable material and slope overhangs should be removed
- Development and maintenance of a mine hazard plan
  - To systematically record hazards (based on observations during regular inspections), as well as reported failures or rock falls
- Regular inspections
  - Slope crests and faces, especially along access ramps and above working areas, should be inspected regularly
  - Any evidence of cracking along slope crests, or potentially unstable blocks on slope faces, should be reported and indicated on the mine hazard plan superimposed on a plan of the current pit layout
  - Significant falls of ground, as well as slope failures, should be reported in detail and indicated on the mine hazard plan. Small-scale toppling and ravelling should be reported, generally in terms of occurrence and location
- Survey monitoring
  - Systematic monitoring should be carried out
  - A variety of instrumentation is available to facilitate detection of both surface and deep-seated slope movement. State-of-the-art monitoring systems currently available can be connected to continuous data recorders, and can be programmed to trigger an alarm when movement exceeds a pre-determined threshold limit
  - Beacons and targets of monitoring systems should be well founded to ensure that observed slope movement is due to slope deformations, and not as a result of instability of the beacons or targets
- Development of an emergency evacuation plan

- Ultimately, to facilitate the extraction of personnel, and if the situation allows, equipment, to safe assembly areas
- The evacuation plan must also define the different levels of emergency situations and cite the appropriate reaction by mine staff (designates when an alarm should be sounded, who should sound the alarm and how mine staff should respond – good communication is a pre-requisite for successful evacuation)

Increasing stack angle from the recommended pit geometry

By increasing the overall stack angle, further provisions to mitigate for the risk of rock falls will be required. In addition to the slope management plan summarised above, further measures to mitigate against the consequences of rock fall may include:

- The implementation of rock fall catch fences / barriers / drapes/ or rock fall netting.
- The implementation of enhanced monitoring systems. Examples include:
  - GeoMos monitoring system: Geodetic measurements are made (at specified intervals) by sensors at a fixed point from targets mounted on the rock face or area of interest. Software processes the data to plot movement vectors. These plots then need to be interpreted by the geotechnical engineer
  - Ground Probe, Reutech or IBIS radar monitoring systems: the system remotely scans the rock face or area of interest using a radar beam. The radar samples the structural displacements and can "detect" very small structural "nonhomogeneities". Software processes the data to plot displacement vectors. These plots then need to be interpreted by the geotechnical engineer

7 Conclusions and Recommendations

7.1 Conclusions

Based on the geotechnical evaluation carried out for the proposed pit, including the geotechnical drilling and logging programme and the laboratory testing, the following conclusions and recommendations can be drawn from the study:

- The joint set orientations defined from the drill core data have been used for the kinematic analysis
- The orientations of brittle structures preclude the likelihood of structurally controlled failures
- The estimation of rock mass strength parameters was based on the Hoek-Brown criterion, with the joint strength parameters assessed using the Barton-Bandis criterion with the relevant adjustments made
- The current proposed pit design has been used as a point of reference for the stability analysis, with sensitivity analyses being carried out to validate the recommended slope angles
- Stability analyses for overall pit slopes have been carried out utilising the limit equilibrium software, SLIDE, and the finite element analysis, PHASE 2 on representative sections of the proposed pit at Otjikoto
- The results of the overall stability analysis confirm acceptable stability conditions for the recommended slope designs
- Due to the recommended steep slope angles a significant risk to the operation is caused by rock fall material, which ultimately may report to the operating levels and injure persons working in the pit and/or damage equipment that is operating in the pit
- The current study considered that the likelihood of rock falls is associated with slope angles – steeper slopes correspond with more frequent rock falls. Furthermore, the higher the frequency of rock falls, the greater the likelihood of material reporting beyond the ramp and in turn, the greater the probability of fatality and/or demolition of equipment to persons and equipment operating in the pit
- Vital aspects emerging from the event tree analysis are:
  - The efficacy of detection of rock falls by visual inspection and instrumentation

  - The efficacy of evacuation
  - The exposure of personnel and/or equipment
- Although access to high risk areas can be restricted to essential personnel only, the focus is on improving the surrounding conditions. This can be achieved by successful limit blasting and good housekeeping practices
- A geotechnical management programme is required for the successful management of the geotechnical risks, and for optimisation of the slope design. This should include the on-going collection of geotechnical and structural data, monitoring of ground water, review of blasting and mining practice, and survey monitoring of the slopes
- Limit blasting techniques should be implemented for both interim and final pit slopes. This will limit damage to the rock mass behind the mine design line and limit back break of benches which could result in rock fall hazards. The author recommends experimentation of limit blasting prior to mining the final slopes, as a poor limit blast at the top of a stack will present a rock fall hazard for the life of that stack
- Following the assessment of the risk profile at the mine, the results were compared to criteria adopted by mining and other engineering disciplines. The existing risks can be reduced by increased slope monitoring capability and by revisions to evacuation procedures. There is also an opportunity to further reduce risks by an increased awareness workforce programme and training.

From the rock mass classification system utilised, the following can be 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. 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.
- 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 and suitable application of each system (Palmstrom, 2008).
- The use of more than one rock mass classification system is suggested as this allows for validation and confirmation of assessments made. This in turn increases confidence in the results achieved.
- The use of classification systems can be of great assistance during the feasibility and preliminary design stages of a project, when little information on the rock mass and its stress and hydrologic environment is available (Palmstrom and Broch, 2006).
- 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.

7.2 Recommendations

Due to the orientations of brittle structures within the rock mass, stack and overall structurally controlled instability is not anticipated. The high rock mass strengths and resulting high factors of safety for overall rock mass stability indicates that rock mass failure is unlikely to occur. The slope design is therefore based on the potential for rock falls to report to working areas, as this is largely dependent on blasting and housekeeping. Optimisation of these angles can be attained by good mining practice and engineered rock fall retardation measures. Double benching should be considered as a practical rock retardation measure.

Figure 7-1 includes a plot showing the east wall of the proposed pit, with the recommended geotechnical berm widths, berm widths and the stack height (toe to crest). Typical profiles with the details of the recommended bench and stack slope configurations are shown in Table 7-1.

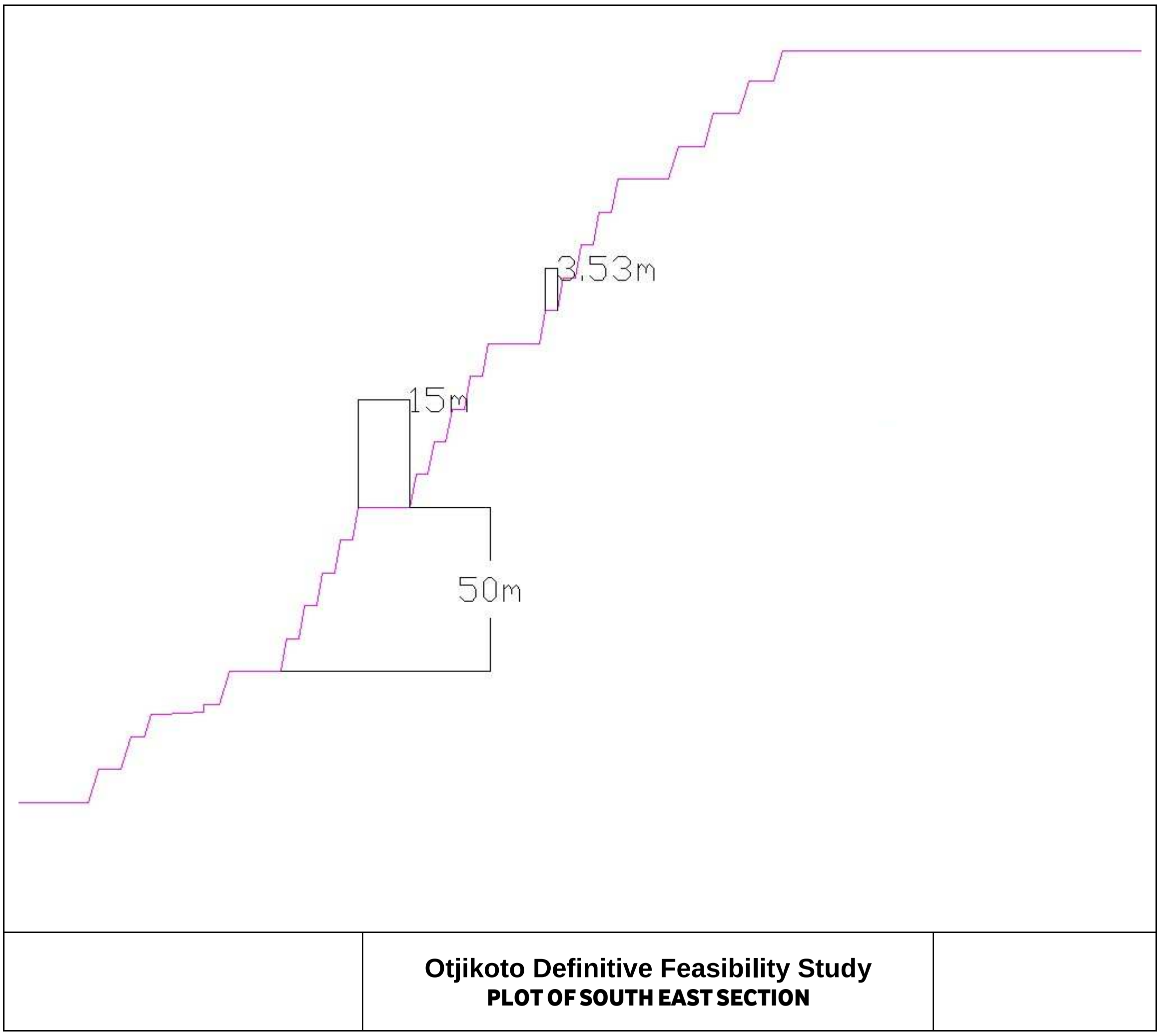


Figure 7-1: Plot showing recommended stack height and berm widths

Table 7-1: Proposed slope configurations for the Eastern Walls and the South West section

	Eastern Walls	**South West**
Overall Angle	50°	47°
Stack Angle	65°	60°
Stack Height	50 m	50 m
Geotechnical Berm Width	15 m	15 m
Bench Face Angle	80°	80°
Bench Height (Single Bench)	10 m	10 m
Berm Width (Single Bench)	3.5 m	4.1 m
Bench Height (Double Bench)	20 m	20 m
Berm Width (Double Bench)	7.0 m	8.2 m

Based on the analysed probabilities of rock fall, the above slope design only falls within acceptable criteria when benches are mined in 20 m lifts.

It is the recommendation of the author that further work be done in the realm of 3-Dimensional modelling, to determine the possibility of steepening the angles in the southern portion of the pit. The author deems this as the next stage of providing insight into the determination of steeper slope angles, with the fairly concentric shape in the southern portion of the pit providing confinement.

8 REFERENCES

Australian Geomechanics Society (AGS). 2007. Guideline for landslide susceptibility, hazard and risk zoning for land use planning. Australian Geomechanics, 42 (1).

Barton, N., Lien, R., Lunde, J. 1974. Engineering Classification of Rock Masses for the Design of Tunnel Support. *Rock Mechanics* 6(4): 189-236.

Barton, N., Loset, F., Lien, R, Lunde, J. 1980. Application of Q-System in Design Decisions concerning Dimensions and Appropriate Support for Underground Installations. *International Symposium on Subsurface Space* 2:553-561.

Barton, N. 1995. The influence of joint properties in modelling joint rock masses. *Proc. International ISRM Congress on Rock Mechanics*. Tokyo, Japan, T, Fujii ed,. A.A. Balkema: Rotterdam. 1023-1032.

Barton, N. 2002. Some new Q-value correlations to assist in site characterisation and tunnel design. *International Journal of Rock Mechanics and Mining Sciences* 39: 185-188.

Bieniawski, Z.T. 1973. Engineering classification of jointed rock masses. *Transactions of the South African Institution of Civil Engineers* 15: 335-343.

Bieniawski, Z.T. 1974. Geomechanics Classification of Rock Masses and its Application in Tunnelling. *Proceedings of the 3rd International Congress on Rock Mechanics, ISRM*. 2(A): 27-32.

Bieniawski, Z.T. 1975. Case Studies: Prediction of Rock Mass Behaviour by the Geomechanics Classification. *Proceedings of the 2nd Austrailian/New-Zealand Conference on Geomechanics.* 36-41.

Bieniawski, Z.T. 1976. Rock Mass Classifications in Rock Engineering. *Proceedings of the symposium on Exploration for Rock Engineering*. 97-105.

Bieniawski, Z.T. 1979. The Geomechanics Classification in Rock Engineering Applications. *Proceedings of the 4th International Congress on Rock Mechanics.* 2: 41-48.

Bieniawski, Z.T. 1989. Engineering Rock Mass Classifications. John Wiley and Sons Inc. Canada. 51-105.

Bieniawski, Z.T. 1990. Tunnel Design by Rock Mass Classifications. Update of Technical Report GL-79-19. Department of the Army. US Army Corps of Engineers. Washington DC.

Brown, T and Booth, A. 2008. “Risk Management”, in Read, J. and Stacey, P. (eds), *Guidelines for Open Pit Slope Design,* CSIRO Publishing.

Cecil, O.S. 1970. Correlation of rock bolt – Shotcrete support and rock quality parameters in Scandinavian tunnels. Ph.D. Thesis. Urbana: University of Illinois.

Chiwaye, H.T. 2010. A Comparison of the Limit Equilibrium and Numerical Modelling Approaches to Risk Analysis for Open Pit Mine Slopes. Master’s Thesis. School of Mining Engineering. University of the Witwatersrand.

Coates, D.F. 1964. Classification of rocks for rock mechanics, Int. J. Rock Mech. Min., vol 1 pp. 421-429, In: Edelbro, C. 2003. Rock mass strength – a review, Department of Civil and Mining Engineering, Division of Rock Mechanics, Luleå University of Technology. pp. 44-73.

Coates, D.F. and Parsons, R.C. 1966. Experimental criteria for classification of rock substances, Int. J. Rock Mech. Min., vol 3 pp. 181-189, In: Edelbro, C. 2003. Rock mass strength – a review, Department of Civil and Mining Engineering, Division of Rock Mechanics, Luleå University of Technology. pp. 44-73.

Deere, D.U. 1964. Technical description of rock cores for engineering purposes. Rock Mechanics Rock Engineering 1. 107-116

Deere, D. U. 1967. Engineering classification and index properties for intact rock, Report No. AFWL-T-65-116. Air Force Weapons Laboratory, Kirtland Air Force Base, New Mexico.

Deere, D.U. 1968. Geological considerations. Chapter 1 in Stagg, K.G. and Zienkiewicz, O.C. (eds), Rock *Mechanics in Engineering Practice,* New York: Wiley. 1-20.

Deere, D.U. and Deere, D.W. 1988. The Rock Quality Designation (RQD) in Practice. *Rock Classification Systems for Engineering Purposes*. ASTM STP 984. 91-101.

Deere, D.U. and Deere, D.W. 1989. Rock Quality Designation after Twenty Years. US Army Corps of Engineers. Contract Report.

Dyke, G.P. 2006. A Quantitative Correlation between the Mining Rock Mass Rating and In-Situ Rock Mass Rating Classification Systems. Master's Thesis. School of Mining Engineering. University of the Witwatersrand.

Edelbro, C. 2003. Rock mass strength- a review. Department of Civil and Mining Engineering. Division of Rock Mechanics. Lulea University of Technology. 44-73

Goel, R.K., Jethwa, J.L., Paithankars, A.G. 1996. Correlation between Barton's Q and Bieniawski's RMR. A New Approach. *International Journal of Rock Mechanics, Mining Sciences and Geomechanics. Abstract* 33(2): 179-181.

Golser, J. 1976. The New Austrain Tunneling Method (NATM), *Theoretical Background and Practical Experiences. 2nd Shotcrete Conference*. 1-6.

Grimstad, E. and Barton N. 1993. Updating of the Q-System for NMT. *Proceedings of the International Symposium on Sprayed Concrete – Modern Use of Wet Mix Sprayed Concrete for Underground Support, Fagernes.* (eds) Kompen, Opsahl and Berg. Norwegian Concrete Association, Oslo.

Hack, H.R.G.K. 1996 Slope stability probability classification. ITC, Enschede, The Netherlands. ISBN 90 6164 125 X. 258pp

Haines A. and Terbrugge P.J. 1991. Preliminary estimate of rock slope stability using rock mass classification systems. In *7th Congress of the International Society of Rock Mechanics,* Aachen, Germany, pp. 887-892.

Health and Safety Executive (HSE). 2001. Reducing risks, protecting people: HSE's decision making process. Suffolk: HSE Books.

Hoek, E and Brown E.T. 1980. Empirical strength criterion for rock masses, Journal Geotechnical Engineering Division. ASCE 106. 1013-1035

Hoek, E. 1994. Strength of Rock and Rock Masses. ISRM News Journal. 2(2): 4-16.

Hoek, E., Kaiser, P.K. and Bawden, W. F. 1995. Support of underground excavations in hard rock, A.A. Balkema/Rotterdam/Brookfield.

Hoek, E. 1997. Practical Estimates of Rock Mass Strength. *International Journal of Rock Mechanics and Mining Sciences*. 34(8): 1165-1186.

Hoek, E. 2007. Practical Rock Engineering. Online Course Notes and Books. https://www.rocscience.com/learning/hoek-s-corner/books

Hoek, E. 2012. Evolution of the Hoek-Brown criterion and the associated Geological Strength Index GSI. Evolution of HB criterion and GSI classification – Second draft. 1-52.

Hoek, E., Carter, T.G. and Diederichs, M.S. 2013. Quantification of the Geological Strength Index Chart. American Rock Mechanics Association 13(672): 1-8.

Hutchinson, D.J. and Diederichs, M.S. 1996. Cablebolting in underground mines. Bitech Publishers Ltd. Canada. 177-208.

Kanda, M.J. and Stacey, T.R. 2016. The influence of various factors on the results of stability analysis of rock slopes and on the evaluation of risk. *The Journal of the Southern African Institute of Mining and Metallurgy*. 116 pp 1075-1081.

Laubscher, D.H. 1975. Class Distinction in Rock Masses. Coal, Gold and Base Minerals of S.A. 23(6): 37-50.

Laubscher, D.H. and Taylor, H.W. 1976. The importance of Geomechanics classification of jointed rock masses in mining operations. *Proceedings of the Symposium on Exploration for Rock Engineering.* 119-128.

Laubscher, D.H. 1977. Geomechanics Classification of Jointed Rock Masses – Mining Applications. *Trans. Institution of Mining and Metallurgy. Section A(*86): A1-A8.

Laubscher, D.H. 1990. A Geomechanics Classification System for the Rating of Rock Mass in Mine Design. *Journal of the South African institute of Mining and Metallurgy* 90(10): 257-473.

Laubscher, D.H. 1994. Cave Mining - The state of the art. The Journal of the South African Institute of Mining and Metallurgy, vol 94, no. 10. pp 279-293.

Laubscher, D.H., Jakubec,J. 2000. The MRMR Rock Mass Classification for Jointed Rock Masses. SME Conference – Underground Mining Methods: Engineering Fundamentals and International Case Studies. 475-481.

Lauffer, H. 1958. Classification for tunnel construction, *Geologie und Bauwesen*. 24(91): 46-51.

Lucian, C. and Wangwe, E.M. 2013. The Usefulness of Rock Quality Designation (RQD) in Determining Strength of Rock. *International Refereed Journal of Engineering and Science.* 9(2) 36-40.

Lorig, L., Stacey, P. and Read, J. 2008. "Slope Design Methods", in Read, J. and Stacey, P. (eds), *Guidelines for Open Pit Slope Design,* CSIRO Publishing.

Marinos, P. and Hoek, E. 2000. A geologically friendly tool for rock mass strength estimation. *Proceedings of the GeoEng2000, International Conference on Geotechnical and Geological Engineering.*1422-1446.

Marinos, V., Marinos, P. and Hoek, E. 2005. The geological strength index: applications and limitations. *Bull Eng Geol Environ.* 64: 55-65

Miller, R. McG., ed. 1983. Evolution of the Damara Orogen of South West Africa/Namibia. Geological Society of South Africa Special Publication 11.

Milne, D., Hadjigeorgiou, J. and Pakalnis, R. (1998) Rock Mass Characterisation for underground hard rock mines. *Tunnelling and Underground Space Technology*. 13(4) 383-391.

Pacher, F., Rabcewicz, L. and Golser, J. 1974. Zum der seitigen stand der gebirgsklassifizierung in stolen-und tunnel. Proc. XXII Geomech. Colloq., Salzburg. 51-58.

Palmstrom, A. 1974. Characterisation of jointing density and the quality of rock masses. Internal report, A.B. Berdal, Norway. 26p.

Palmstrom, A. 1982. The volumetric joint count – A useful and simple measure of the degree of rock mass jointing. IAEG Congress, New Delhi, 1982. pp. V.221-V.228.

Palmstrom, A. 2005. Measurements of and Correlations between Block Size and Rock Quality Designation (RQD). *Tunnels and Underground Space Technology* 20: 362-377.

Palmstrom, A and Broch, E. 2006. Use and Misuse of Rock Mass Classification Systems with particular reference to the Q-System. *Tunnels and Underground Space Technology* 21: 575-593.

Patching, T.H. and Coates, D.F. 1968. A recommended rock classification for rock mechanic purposes, CIM Bulletin. Pp. 1195-1197

Rathod, G.W. and Rao, K.S. 2012. Finite element and reliability analyses for slope stability of Subansiri Lower Hydroelectric Project: a case study. *Geotechnical and Geological Engineering,* vol. 30. pp 233-252.

Read, J. and Stacey, P. 2008. Guidelines for Open Pit Slope Design. CSIRO Publishing.

Singh, B. and Goel, R.K. 1999. Rock mass classification – A practical approach in civil engineering. Elsevier Amsterdam/New York/Oxford. 1-99

SRK Project Report, 2006. The Stability of Rock Slopes in Large Open Pit Mines Design Manual. Report prepared for CSIRO Research Project Number: 2004-ha62

SRK Project Report, 2010.Independent Technical Report on the Otjikoto Gold Project. Report prepared for 0824239 BC Limited (Auryx Gold Corporation) Project Number: 416229

Steffen, O.K.H., Contreras, L.F., Terbrugge, P.J. and Venter, J. 2008. A Risk Evaluation Approach for Pit Slope Design. 42nd US Rock Mechanics Symposium and 2nd US-Canada Rock Mechanics Symposium, held in San Francisco, June 29 - July 2, 2008.

Steffen, O.K.H. 1997. Planning of open pit mines on a risk basis. *Journal of South African Institute of Mining and Metallurgy*, vol. 97, no. 2, pp. 47-56. , March/April 1997.

Steffen, O.K.H., Terbrugge, P.J., Wesseloo, J. and Venter, J. 2006. A Risk Consequence Approach to Open Pit Slope Design, SAIMM International Symposium on Stability of Rock Slopes in Open Pit Mining and Civil Engineering.

Stini, J. 1950. Tunnelbaugeologie. Springer. Vienna. 366pp.

Tapia, A., Contreras, L.F., Jefferies, M.G. and Steffen, O. 2007. Risk Evaluation of slope failure at Chuquicamata Mine. *In: Proceedings Int. Symp. Rock Slope Stability in Open Pit Mining and Civil Engineering*. 12-14 September 2007, Perth, Australia. Ed. Yves Potvin, Australian Centre for Geomechanics, Perth, Australia. pp. 477-485

Terzaghi, K. 1946. Rock defects and loads on tunnel supports. Rock tunnelling with Steel Supports,ed. Proctor, R. V. and White, T. Commercial Shearing Co. 15-99.

Ureel, S. and Momayez, M. 2014. An investigation of the limit equilibrium method and numerical modelling for rock slope stability analysis. *Rock Mechanics and its Applications in Civil, Mining and Petroleum Engineering. Proceedings of the Geo-Shanghai 2014 International Conference,* Shanghai, China, 26-28 May 2014. Zhang, L. and Wong, E.L.N.Y. (eds) Geotechnical Special Publication 237. American Society of Civil Engineers. pp 218-227.

US Army Corps of Engineers. 1994. Engineering and design – rock foundations, publication number: EM 1110-1-2908. Chapter 4. 4-1 - 4-9

van der Merwe, A.J. and Jones, C. 2005. Otavi Exploration Project, Namibia, Independent Technical Report. RSG Global (16 September 2005.

Wickham, G. E., Tiedeman, H. R. and Skinner, E. H. 1972. Support determination based on geological predictions*. North American Rapid Excavation and Tunneling Conference*. 1: 43-64.

Wilson, R. and Crouch, E.A.C. 1987. Risk Assessment and Comparisons: An Introduction. Science, Vol. 236. pp 267-270

Appendices

Appendix A: Rock Mass Classification

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
OT20	1	0	10	10	ALB	135.8	25.5	25.3	64.9	60.8	8.0
OT24	2	10	20	10	ALB	154.5	24.4	22.2	62.6	55.5	8.0
OT24	4	20	40	20	ALB	285.0	51.5	44.7	125.8	115.3	16.0
OT26	5	40	50	10	ALB	154.5	21.2	23.4	60.7	53.0	8.0
OT26	6	50	60	10	ALB	105.5	33.2	23.7	68.0	68.8	8.0
OT26	8	60	80	20	ALB	269.0	48.9	38.9	115.8	104.2	16.0
OT27	11	80	110	30	ALB	458.7	74.4	32.8	154.9	125.2	24.0
OT27	13	110	130	20	ALB	382.9	49.3	24.4	113.4	86.4	16.0
OT31	14	130	140	10	ALB	114.6	36.5	18.4	66.9	66.1	8.0
OT31	15	140	150	10	ALB	43.1	23.9	19.3	48.5	51.3	8.0
OT36	16	150	160	10	ALB	90.0	23.1	19.1	52.1	50.0	8.0
OT36	17	160	170	10	ALB	127.4	27.0	14.6	54.9	48.8	8.0
OT51	20	170	200	30	ALB	463.5	71.1	61.1	180.2	156.9	24.0
OT51	21	200	210	10	ALB	138.9	26.8	22.8	64.0	59.2	8.0
OT51	22	210	220	10	ALB	151.9	26.5	20.4	62.6	55.8	8.0
OT53	23	220	230	10	ALB	154.5	25.7	20.4	62.1	54.9	8.0
OT57	25	230	250	20	ALB	242.2	45.6	42.7	113.9	104.8	16.0
OT57	26	250	260	10	ALB	117.1	26.0	24.1	62.5	60.0	8.0
GT1	27	260	270	10	ALB	52.7	23.5	25.0	54.7	57.7	8.0
GT1	28	270	280	10	ALB	98.7	18.4	24.5	53.6	50.6	8.0
GT1	29	280	290	10	ALB	101.4	22.6	22.2	55.7	53.3	8.0
GT3	30	290	300	10	ALB	83.8	22.5	22.4	54.2	53.3	8.0
GT3	31	300	310	10	ALB	148.8	20.4	21.3	57.1	49.3	8.0
GT3	32	310	320	10	ALB	71.2	17.5	24.5	50.0	49.4	8.0
GT4	33	320	330	10	ALB	136.8	20.6	20.9	55.8	49.0	8.0
GT4	34	330	340	10	ALB	39.5	23.5	17.9	46.3	49.0	8.0
TC184	35	340	350	10	ALB	94.5	24.0	21.1	55.4	53.5	8.0
TC184	36	350	360	10	ALB	143.5	18.3	23.5	56.7	49.3	8.0
OTGT12-006	37	360	370	10	ALB	188.7	30.9	23.6	74.1	65.7	8.0
OTGT12-006	38	370	380	10	ALB	192.5	31.9	21.8	73.7	64.5	8.0
OTGT12-009	39	380	390	10	ALB	192.5	32.8	23.0	75.8	67.5	8.0
OTGT12-009	40	390	400	10	ALB	192.5	33.2	22.0	75.2	66.5	8.0
OTGT12-012	41	400	410	10	ALB	121.7	33.6	24.0	70.3	69.7	8.0
OTGT12-012	42	410	420	10	ALB	192.5	40.0	40.0	100.0	100.0	8.0
OTGT12-012	43	420	430	10	ALB	192.5	40.0	35.7	95.7	94.1	8.0
OTGT12-012	44	430	440	10	ALB	192.5	32.4	27.9	80.2	73.4	8.0
OTGT12-012	45	440	450	10	ALB	192.5	30.4	27.5	77.9	70.3	8.0
OTGT12-012	46	450	460	10	ALB	192.5	40.0	40.0	100.0	100.0	8.0
			TOTAL	**460**							

		IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
	n	46	46	46	46	46	46
ALB	**Average**	139.4	26.5	21.9	63.0	57.9	8.0
	Avg - Stdv	97.7	20.9	15.8	50.6	44.5	8.0
	Avg + Stdv	181.0	32.0	27.9	75.4	71.2	8.0
	Stdv	41.7	5.6	6.0	12.4	13.4	0.0
	CV	0.30	0.21	0.28	0.20	0.23	0.00

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
OT45	1	0	10	10	BS	177.1	21.1	19.9	59.4	48.5	7.0
OT56	4	10	40	30	BS	553.1	64.5	29.8	151.8	110.5	21.0
GT2	5	40	50	10	BS	154.5	24.4	15.4	55.8	47.0	7.0
GT2	6	50	60	10	BS	95.2	18.5	21.7	50.5	47.4	7.0
GT2	7	60	70	10	BS	69.9	24.1	22.1	54.1	54.9	7.0
GT4	8	70	80	10	BS	101.9	16.9	22.2	50.0	46.0	7.0
OTGT12-008	9	80	90	10	BS	99.1	26.4	24.0	60.9	60.2	7.0
OTGT12-008	10	90	100	10	BS	97.2	27.3	22.5	60.1	59.6	7.0
OTGT12-009	11	100	110	10	BS	88.2	33.7	22.6	65.7	67.9	7.0
OTGT12-009	12	110	120	10	BS	192.5	30.8	21.0	71.8	62.1	7.0
OTGT12-010	13	120	130	10	BS	192.5	37.6	30.4	88.0	83.7	7.0
OTGT12-010	14	130	140	10	BS	192.5	34.6	29.7	84.3	78.9	7.0
OTGT12-011	15	140	150	10	BS	192.5	32.8	22.0	74.8	66.1	7.0
OTGT12-011	16	150	160	10	BS	192.5	33.9	21.5	75.5	66.7	7.0
OTGT12-011	17	160	170	10	BS	192.5	39.5	34.2	93.7	91.4	7.0
OTGT12-011	18	170	180	10	BS	192.5	27.9	24.1	71.9	62.3	7.0
OTGT12-011	19	180	190	10	BS	192.5	30.1	25.6	75.7	67.3	7.0
OTGT12-011	20	190	200	10	BS	192.5	31.8	22.5	74.3	65.3	7.0
OTGT12-011	21	200	210	10	BS	192.5	36.7	29.4	86.1	81.1	7.0
OTGT12-012	22	210	220	10	BS	132.6	36.2	27.4	77.5	77.7	7.0
OTGT12-012	23	220	230	10	BS	74.5	32.3	22.5	62.8	65.8	7.0
OTGT12-012	24	230	240	10	BS	122.6	31.1	29.9	73.9	74.5	7.0
OTGT12-012	25	240	250	10	BS	192.5	38.1	29.9	88.1	83.7	7.0
OTGT12-012	26	250	260	10	BS	192.5	35.0	25.6	80.6	73.7	7.0
OTGT12-012	27	260	270	10	BS	192.5	34.0	29.4	83.4	77.7	7.0
OTGT12-012	28	270	280	10	BS	192.5	31.7	20.5	72.2	62.5	7.0
			TOTAL	**280**							

		IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
	n	28	28	28	28	28	28
BS	**Average**	159.3	29.7	23.1	69.4	63.7	7.0
BS	**Avg - Stdv**	114.2	23.3	16.9	56.0	48.6	7.0
BS	**Avg + Stdv**	204.4	36.1	29.3	82.8	78.7	7.0
	Stdv	45.1	6.4	6.2	13.4	15.1	0.0
	CV	0.28	0.21	0.27	0.19	0.24	0.00

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
OT20	1	0	10	10	Calc	54.5	24.0	12.9	43.0	43.5	9.0
OT24	2	10	20	10	Calc	54.5	21.2	10.8	38.0	37.4	9.0
OT26	3	20	30	10	Calc	54.5	19.7	13.9	39.5	39.5	9.0
OT31	4	30	40	10	Calc	49.9	22.8	31.9	60.4	65.5	9.0
OT31	5	40	50	10	Calc	57.3	21.0	23.4	50.7	52.6	9.0
OT36	6	50	60	10	Calc	54.5	23.4	14.4	43.8	44.5	9.0
OT45	7	60	70	10	Calc	54.5	29.9	24.0	59.9	64.9	9.0
OT51	8	70	80	10	Calc	35.5	26.0	15.6	45.8	49.0	9.0
OT51	9	80	90	10	Calc	54.5	21.7	28.0	55.6	58.9	9.0
OT56	10	90	100	10	Calc	18.0	24.0	33.5	60.4	69.4	9.0
TC115	12	100	120	20	Calc	95.6	38.6	42.9	92.5	96.3	18.0
OTGT12-005	13	120	130	10	Calc	12.9	26.1	26.9	55.2	63.9	9.0
OTGT12-006	14	130	140	10	Calc	60.9	23.9	21.7	52.3	54.1	9.0
OTGT12-007	15	140	150	10	Calc	51.1	25.9	23.0	54.7	58.5	9.0
OTGT12-008	16	150	160	10	Calc	52.2	25.7	21.7	53.2	56.5	9.0
OTGT12-009	17	160	170	10	Calc	31.9	26.9	22.5	53.6	59.0	9.0
OTGT12-009	18	170	180	10	Calc	47.9	34.4	21.1	60.8	66.7	9.0
OTGT12-012	19	180	190	10	Calc	18.4	28.7	24.1	55.7	63.4	9.0
OTGT12-012	20	190	200	10	Calc	29.5	32.5	23.3	59.8	67.3	9.0
			TOTAL	**200**							

		IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
	n	20	20	20	20	20	20
Calc	**Average**	44.4	24.8	21.8	51.7	55.5	9.0
	Avg - Stdv	29.8	20.6	15.8	44.5	45.8	9.0
	Avg + Stdv	59.0	29.0	27.7	59.0	65.3	9.0
	Stdv	14.6	4.2	6.0	7.2	9.8	0.0
	CV	0.33	0.17	0.27	0.14	0.18	0.00

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
OT27	1	0	10	10	FW	98.6	34.6	24.6	69.8	71.8	14.0
OTGT12-006	2	10	20	10	FW	153.6	34.7	22.8	73.6	69.7	14.0
OTGT12-007	3	20	30	10	FW	192.5	33.7	28.8	82.4	76.2	14.0
OTGT12-012	4	30	40	10	FW	135.5	28.9	21.1	64.3	59.9	14.0
OTGT12-012	5	40	50	10	FW	192.5	37.0	20.5	77.5	69.2	14.0
			TOTAL	**50**							

		IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
	n	5	5	5	5	5	5
FW	**Average**	154.5	33.8	23.6	73.5	69.3	14.0
FW	**Avg - Stdv**	114.6	30.8	20.2	66.6	63.4	14.0
FW	**Avg + Stdv**	194.5	36.8	26.9	80.5	75.3	14.0
	Stdv	39.9	3.0	3.3	6.9	6.0	0.0
	CV	0.26	0.09	0.14	0.09	0.09	0.00

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
OT20	2	0	20	20	HF	303.5	44.3	25.4	101.2	81.9	28.0
OT20	3	20	30	10	HF	134.1	26.9	24.0	64.9	60.9	14.0
OT20	4	30	40	10	HF	169.7	22.0	21.8	61.4	52.0	14.0
OT20	5	40	50	10	HF	154.5	21.0	10.0	47.0	36.2	14.0
OT20	6	50	60	10	HF	125.0	22.3	7.5	42.9	34.7	14.0
OT20	7	60	70	10	HF	143.9	24.9	18.9	58.8	52.4	14.0
OT20	10	70	100	30	HF	486.7	65.3	35.1	150.8	117.9	42.0
OT24	11	100	110	10	HF	168.0	28.8	24.2	70.5	63.8	14.0
OT24	12	110	120	10	HF	185.1	25.6	22.3	67.1	57.1	14.0
OT24	13	120	130	10	HF	175.2	26.7	21.7	66.6	57.8	14.0
OT24	16	130	160	30	HF	359.9	72.0	72.7	182.4	172.5	42.0
OT26	17	160	170	10	HF	155.5	23.5	13.2	52.8	42.7	14.0
OT26	18	170	180	10	HF	154.5	26.1	14.9	57.0	48.3	14.0
OT26	20	180	200	20	HF	309.0	50.5	38.5	121.0	105.7	28.0
OT26	21	200	210	10	HF	154.5	18.0	22.0	56.0	47.0	14.0
OT27	22	210	220	10	HF	172.5	21.4	21.7	61.0	51.0	14.0
OT31	24	220	240	20	HF	385.0	48.1	33.0	121.1	95.9	28.0
OT31	25	240	250	10	HF	125.0	23.4	12.8	49.3	42.5	14.0
OT36	26	250	260	10	HF	119.5	26.0	22.4	60.9	57.8	14.0
OT36	27	260	270	10	HF	145.6	23.7	24.4	63.4	57.5	14.0
OT36	28	270	280	10	HF	74.3	21.0	20.1	49.4	48.6	14.0
OT45	30	280	300	20	HF	274.4	43.0	49.1	120.7	109.4	28.0
OT45	32	300	320	20	HF	333.4	46.1	48.9	129.5	113.1	28.0
OT51	33	320	330	10	HF	138.7	24.1	21.0	59.5	53.5	14.0
OT56	34	330	340	10	HF	151.1	25.3	24.7	65.7	59.9	14.0
OT56	36	340	360	20	HF	292.8	51.8	49.9	132.1	121.9	28.0
OT56	37	360	370	10	HF	144.1	27.5	17.1	59.5	52.6	14.0
OT56	39	370	390	20	HF	275.4	47.2	60.8	136.7	129.5	28.0
OT57	41	390	410	20	HF	298.5	47.6	57.4	136.0	125.9	28.0
OT57	42	410	420	10	HF	118.8	22.9	24.0	59.3	55.8	14.0
OT57	44	420	440	20	HF	264.3	47.3	48.0	122.9	113.6	28.0
GT1	46	440	460	20	HF	267.7	46.7	45.5	120.3	109.8	28.0
GT1	47	460	470	10	HF	55.7	23.0	25.6	55.1	57.9	14.0
GT1	48	470	480	10	HF	59.2	19.2	25.0	51.0	52.1	14.0
GT1	49	480	490	10	HF	63.3	20.9	23.0	51.2	52.1	14.0
GT1	50	490	500	10	HF	86.1	24.5	19.6	53.5	52.6	14.0
GT1	51	500	510	10	HF	39.5	22.4	14.2	41.6	43.1	14.0
GT1	52	510	520	10	HF	71.7	24.1	16.1	48.4	47.6	14.0
GT3	53	520	530	10	HF	102.6	21.9	24.5	57.5	55.2	14.0
GT3	54	530	540	10	HF	102.1	20.3	27.1	58.4	56.2	14.0
GT3	55	540	550	10	HF	55.8	18.8	19.2	44.6	44.9	14.0
GT3	56	550	560	10	HF	123.9	20.7	25.2	59.0	54.4	14.0
GT3	57	560	570	10	HF	108.4	14.4	17.9	43.9	38.1	14.0
GT3	58	570	580	10	HF	81.0	16.4	22.8	48.2	46.1	14.0
GT3	59	580	590	10	HF	108.7	22.4	24.1	58.2	55.5	14.0
GT3	60	590	600	10	HF	53.6	21.1	22.3	49.8	51.5	14.0
GT3	61	600	610	10	HF	108.8	24.3	24.2	60.2	57.9	14.0
GT3	62	610	620	10	HF	39.5	25.5	26.6	57.2	62.6	14.0
GT3	63	620	630	10	HF	39.5	23.6	29.1	57.7	63.1	14.0
GT3	64	630	640	10	HF	39.5	24.0	27.5	56.6	61.7	14.0
GT4	65	640	650	10	HF	139.4	18.4	23.4	56.4	49.3	14.0
GT4	66	650	660	10	HF	39.5	22.4	30.4	57.8	63.1	14.0
GT4	67	660	670	10	HF	100.0	16.1	25.4	52.2	48.8	14.0
GT4	68	670	680	10	HF	67.8	19.8	23.4	50.9	51.1	14.0
TC115	69	680	690	10	HF	39.5	17.9	22.2	45.1	47.2	14.0
TC115	70	690	700	10	HF	137.1	25.4	26.3	66.1	61.9	14.0
TC115	71	700	710	10	HF	162.8	23.2	18.6	58.7	49.5	14.0
TC115	72	710	720	10	HF	170.7	21.0	27.4	66.2	57.7	14.0
TC115	73	720	730	10	HF	158.5	20.6	24.9	61.9	53.9	14.0
TC115	74	730	740	10	HF	154.5	24.1	20.8	60.9	53.3	14.0

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
TC115	75	740	750	10	HF	158.8	24.9	24.8	66.2	59.4	14.0
TC115	76	750	760	10	HF	167.9	25.0	24.1	66.5	58.6	14.0
TC184	77	760	770	10	HF	137.0	16.9	27.0	58.3	51.6	14.0
TC184	78	770	780	10	HF	68.4	21.3	26.6	55.7	57.0	14.0
TC184	79	780	790	10	HF	115.1	23.1	26.0	61.3	58.4	14.0
TC184	80	790	800	10	HF	123.5	22.3	23.1	58.4	54.0	14.0
TC184	81	800	810	10	HF	39.5	18.3	20.4	43.7	45.6	14.0
TC184	82	810	820	10	HF	101.9	19.9	23.0	53.9	50.6	14.0
OT281	83	820	830	10	HF	41.8	22.3	25.6	53.2	57.1	14.0
OT281	84	830	840	10	HF	39.5	24.6	28.2	57.8	63.3	14.0
OT281	85	840	850	10	HF	39.5	23.2	24.8	53.0	57.2	14.0
OT281	86	850	860	10	HF	54.9	28.9	25.6	61.0	65.7	14.0
OT281	87	860	870	10	HF	88.0	25.0	25.2	59.8	60.0	14.0
OT281	88	870	880	10	HF	39.5	25.0	23.8	53.8	58.3	14.0
OT281	89	880	890	10	HF	39.5	20.9	24.1	50.1	53.4	14.0
OT281	90	890	900	10	HF	39.5	21.6	23.7	50.3	53.7	14.0
OT281	91	900	910	10	HF	39.5	18.5	18.9	42.5	44.0	14.0
OTGT12-006	92	910	920	10	HF	110.9	27.8	21.3	60.9	59.0	14.0
OTGT12-006	93	920	930	10	HF	192.5	29.7	24.4	74.1	65.3	14.0
OTGT12-007	94	930	940	10	HF	134.4	31.8	25.7	71.6	70.3	14.0
OTGT12-007	95	940	950	10	HF	144.4	28.7	24.1	67.9	63.6	14.0
OTGT12-008	96	950	960	10	HF	192.5	34.9	24.0	78.8	71.3	14.0
OTGT12-010	97	960	970	10	HF	104.5	28.9	20.5	60.5	59.0	14.0
OTGT12-010	98	970	980	10	HF	60.5	25.2	20.5	52.2	54.3	14.0
OTGT12-010	99	980	990	10	HF	71.6	32.6	22.0	62.3	65.7	14.0
OTGT12-010	100	990	1000	10	HF	187.1	27.9	20.5	67.8	57.7	14.0
OTGT12-010	101	1000	1010	10	HF	192.5	29.6	20.9	70.5	60.2	14.0
OTGT12-010	102	1010	1020	10	HF	192.5	39.3	33.6	92.9	90.3	14.0
OTGT12-010	103	1020	1030	10	HF	192.5	32.2	28.8	81.0	74.6	14.0
OTGT12-010	104	1030	1040	10	HF	171.2	32.9	22.0	72.8	65.9	14.0
OTGT12-011	105	1040	1050	10	HF	54.5	27.9	20.5	54.4	57.7	14.0
OTGT12-011	106	1050	1060	10	HF	54.5	23.3	20.5	49.8	51.9	14.0
OTGT12-011	107	1060	1070	10	HF	66.5	24.9	20.5	52.5	53.9	14.0
OTGT12-011	108	1070	1080	10	HF	71.9	28.0	20.5	56.2	57.8	14.0
OTGT12-011	109	1080	1090	10	HF	82.8	36.8	20.5	66.1	68.9	14.0
OTGT12-011	110	1090	1100	10	HF	192.5	33.3	23.6	76.8	68.7	14.0
OTGT12-011	111	1100	1110	10	HF	155.2	27.0	22.0	65.2	58.6	14.0
OTGT12-012	112	1110	1120	10	HF	56.2	29.5	20.5	56.2	59.8	14.0
OTGT12-012	113	1120	1130	10	HF	74.5	33.8	31.9	73.7	81.0	14.0
			TOTAL	**1130**							

		IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
	n	113	113	113	113	113	113
HF	**Average**	118.2	24.2	22.4	59.2	55.6	14.0
HF	**Avg - Stdv**	69.9	20.0	17.7	50.6	46.9	14.0
HF	**Avg + Stdv**	166.6	28.4	27.1	67.7	64.3	14.0
	Stdv	48.3	4.2	4.7	8.6	8.7	0.0
	CV	0.41	0.17	0.21	0.14	0.16	0.00

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
OT26	1	0	10	10	OTB	125.7	23.2	20.2	56.5	51.7	13.0
OT31	3	10	30	20	OTB	215.2	55.0	48.0	125.6	123.5	26.0
OT57	4	30	40	10	OTB	93.3	33.0	23.9	66.8	68.8	13.0
GT1	5	40	50	10	OTB	63.5	27.0	20.8	55.1	57.1	13.0
TC184	6	50	60	10	OTB	68.5	27.7	20.4	55.9	57.5	13.0
OTGT12-007	7	60	70	10	OTB	160.2	35.6	24.6	77.1	73.2	13.0
OTGT12-012	8	70	80	10	OTB	192.5	29.1	25.6	74.7	66.0	13.0
			TOTAL	**80**							

		IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
	n	8	8	8	8	8	8
OTB	**Average**	114.9	28.8	22.9	64.0	62.2	13.0
OTB	**Avg - Stdv**	70.9	25.0	20.8	55.6	55.3	13.0
OTB	**Avg + Stdv**	158.9	32.7	25.1	72.4	69.2	13.0
	Stdv	44.0	3.8	2.1	8.4	7.0	0.0
	CV	0.38	0.13	0.09	0.13	0.11	0.00

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
OT20	1	0	10	10	SCH	120.2	26.8	21.8	61.2	58.2	15.0
OT24	2	10	20	10	SCH	190.0	22.1	15.0	56.9	43.8	15.0
OT27	3	20	30	10	SCH	176.0	21.0	21.1	60.4	49.8	15.0
OT27	5	30	50	20	SCH	345.4	42.0	38.4	116.2	95.0	30.0
OT27	6	50	60	10	SCH	160.0	18.4	20.2	55.2	45.5	15.0
OT31	7	60	70	10	SCH	106.5	18.1	16.1	45.5	40.3	15.0
OT31	9	70	90	20	SCH	371.7	41.7	24.0	104.4	77.0	30.0
OT45	11	90	110	20	SCH	143.2	37.1	25.5	78.7	73.6	30.0
OT45	13	110	130	20	SCH	277.2	42.0	32.1	102.9	87.3	30.0
OT45	14	130	140	10	SCH	154.5	22.0	25.6	63.6	56.6	15.0
OT45	17	140	170	30	SCH	440.0	78.1	75.0	198.7	183.3	45.0
OT53	18	170	180	10	SCH	192.5	24.6	20.9	65.5	54.1	15.0
OT53	20	180	200	20	SCH	385.0	51.2	41.0	132.2	109.7	30.0
OT56	23	200	230	30	SCH	548.4	68.3	72.9	198.1	168.2	45.0
OT56	24	230	240	10	SCH	154.5	20.9	23.3	60.2	52.4	15.0
OT57	25	240	250	10	SCH	180.8	23.3	24.0	66.0	56.3	15.0
OT57	26	250	260	10	SCH	192.5	23.1	24.0	67.1	56.1	15.0
OT57	27	260	270	10	SCH	192.5	22.2	23.5	65.6	54.2	15.0
OT57	28	270	280	10	SCH	137.4	23.6	20.5	58.5	52.3	15.0
GT1	29	280	290	10	SCH	110.3	18.8	19.2	49.8	44.9	15.0
GT1	30	290	300	10	SCH	39.5	30.4	23.0	58.4	64.4	15.0
GT1	31	300	310	10	SCH	39.5	23.3	20.0	48.3	51.3	15.0
GT2	32	310	320	10	SCH	55.6	20.6	23.7	50.8	52.3	15.0
GT2	33	320	330	10	SCH	154.5	22.6	25.6	64.2	57.4	15.0
GT2	34	330	340	10	SCH	154.5	20.6	30.0	66.6	60.2	15.0
GT2	35	340	350	10	SCH	118.4	16.6	25.6	54.7	49.5	15.0
GT2	36	350	360	10	SCH	39.5	18.1	25.6	48.7	51.5	15.0
GT2	37	360	370	10	SCH	39.5	18.6	25.6	49.2	52.1	15.0
GT2	38	370	380	10	SCH	111.8	22.1	19.2	53.2	48.9	15.0
GT2	39	380	390	10	SCH	154.5	18.5	23.3	57.8	49.2	15.0
GT2	40	390	400	10	SCH	154.5	15.5	22.2	53.7	44.0	15.0
GT2	41	400	410	10	SCH	154.5	17.6	21.3	54.9	45.7	15.0
GT2	42	410	420	10	SCH	135.9	18.9	22.0	55.2	48.2	15.0
GT3	43	420	430	10	SCH	39.5	19.4	25.6	50.0	53.2	15.0
GT3	44	430	440	10	SCH	31.5	24.6	25.6	54.5	60.0	15.0
GT3	45	440	450	10	SCH	78.9	22.1	23.0	53.8	53.4	15.0
GT3	46	450	460	10	SCH	100.1	26.7	19.0	56.5	54.3	15.0
GT3	47	460	470	10	SCH	154.5	24.0	16.9	56.8	48.4	15.0
GT4	48	470	480	10	SCH	154.5	18.8	18.3	53.1	43.6	15.0

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
GT4	49	480	490	10	SCH	151.6	19.3	29.0	64.0	57.2	15.0
GT4	50	490	500	10	SCH	39.5	15.4	21.5	41.8	43.1	15.0
GT4	51	500	510	10	SCH	146.8	20.2	24.5	60.0	53.0	15.0
GT4	52	510	520	10	SCH	51.1	18.5	22.5	47.1	48.4	15.0
GT4	53	520	530	10	SCH	152.5	22.7	19.1	57.6	49.4	15.0
GT4	54	530	540	10	SCH	154.5	21.4	20.8	58.2	49.9	15.0
GT4	55	540	550	10	SCH	92.6	15.0	14.3	39.4	34.6	15.0
GT4	56	550	560	10	SCH	91.4	18.3	18.7	47.0	43.6	15.0
GT4	57	560	570	10	SCH	57.6	19.3	17.5	43.6	43.4	15.0
GT4	58	570	580	10	SCH	83.1	21.7	20.8	51.6	50.3	15.0
GT4	59	580	590	10	SCH	154.5	15.8	20.8	52.6	42.9	15.0
GT4	60	590	600	10	SCH	35.0	23.3	21.3	49.2	53.1	15.0
TC115	61	600	610	10	SCH	25.0	23.7	23.4	50.7	56.1	15.0
TC115	62	610	620	10	SCH	136.0	27.8	22.4	64.6	60.0	15.0
TC115	63	620	630	10	SCH	179.4	18.0	24.1	60.7	49.6	15.0
TC115	64	630	640	10	SCH	154.5	25.1	25.5	66.6	60.4	15.0
TC115	65	640	650	10	SCH	154.5	28.0	24.5	68.6	63.1	15.0
TC115	66	650	660	10	SCH	174.0	21.6	16.7	56.3	45.2	15.0
TC115	67	660	670	10	SCH	162.9	19.6	13.6	50.1	39.1	15.0
TC115	68	670	680	10	SCH	168.3	23.5	17.4	58.4	48.4	15.0
TC115	69	680	690	10	SCH	161.4	23.5	22.9	63.2	55.2	15.0
TC115	70	690	700	10	SCH	128.2	25.3	20.7	59.5	54.8	15.0
TC115	71	700	710	10	SCH	134.3	19.4	13.2	46.6	38.3	15.0
TC115	72	710	720	10	SCH	174.8	27.1	18.7	64.0	54.5	15.0
TC115	73	720	730	10	SCH	129.2	20.5	28.7	62.8	58.3	15.0
TC115	74	730	740	10	SCH	110.6	20.4	25.3	57.6	54.3	15.0
TC184	75	740	750	10	SCH	49.3	18.5	28.8	53.2	55.8	15.0
TC184	76	750	760	10	SCH	46.6	18.3	23.8	47.8	49.7	15.0
TC184	77	760	770	10	SCH	39.5	22.1	18.7	45.7	48.2	15.0
TC184	78	770	780	10	SCH	48.5	22.6	23.3	51.7	54.4	15.0
TC184	79	780	790	10	SCH	126.1	18.3	20.0	51.5	44.9	15.0
TC184	80	790	800	10	SCH	137.8	21.9	21.5	57.8	51.5	15.0
OT281	81	800	810	10	SCH	52.0	23.3	26.2	55.7	59.1	15.0
OT281	82	810	820	10	SCH	39.5	20.6	19.5	45.1	47.3	15.0
OT281	83	820	830	10	SCH	39.5	23.6	19.0	47.5	50.4	15.0
OT281	84	830	840	10	SCH	86.1	22.3	23.9	55.6	54.9	15.0
OT281	85	840	850	10	SCH	51.5	18.0	18.5	42.7	43.0	15.0
OT281	86	850	860	10	SCH	91.1	20.1	22.8	52.9	50.9	15.0
OT281	87	860	870	10	SCH	138.7	24.7	23.7	62.9	57.8	15.0
OT281	88	870	880	10	SCH	45.9	22.0	20.7	48.3	50.6	15.0
OT281	89	880	890	10	SCH	84.0	18.6	20.1	47.9	45.7	15.0
OT281	90	890	900	10	SCH	88.1	20.0	13.9	43.5	39.8	15.0
OT281	91	900	910	10	SCH	39.5	22.0	21.6	48.5	51.7	15.0
OT281	92	910	920	10	SCH	39.5	19.5	19.9	44.3	46.5	15.0
OTGT12-005	93	920	930	10	SCH	68.0	22.1	23.8	53.4	54.7	15.0
OTGT12-005	94	930	940	10	SCH	74.5	20.1	21.8	49.9	49.6	15.0
OTGT12-005	95	940	950	10	SCH	160.6	22.9	20.6	60.2	51.7	15.0
OTGT12-005	96	950	960	10	SCH	192.5	31.6	19.6	71.2	61.3	15.0
OTGT12-005	97	960	970	10	SCH	192.5	21.5	23.2	64.7	53.0	15.0
OTGT12-005	98	970	980	10	SCH	192.5	27.8	24.1	71.8	62.2	15.0
OTGT12-005	99	980	990	10	SCH	192.5	24.9	17.9	62.8	50.8	15.0

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
OTGT12-005	100	990	1000	10	SCH	192.5	27.6	22.0	69.6	59.7	15.0
OTGT12-005	101	1000	1010	10	SCH	192.5	28.6	26.8	75.4	66.9	15.0
OTGT12-005	102	1010	1020	10	SCH	192.5	25.9	25.6	71.5	61.8	15.0
OTGT12-005	103	1020	1030	10	SCH	192.5	25.1	25.6	70.7	60.7	15.0
OTGT12-005	104	1030	1040	10	SCH	192.5	35.2	25.6	80.8	74.0	15.0
OTGT12-006	105	1040	1050	10	SCH	130.0	29.8	25.6	69.1	66.9	15.0
OTGT12-006	106	1050	1060	10	SCH	150.7	30.9	23.3	70.0	65.4	15.0
OTGT12-006	107	1060	1070	10	SCH	192.5	38.1	29.9	88.0	83.7	15.0
OTGT12-007	108	1070	1080	10	SCH	192.5	32.6	23.9	76.5	68.2	15.0
OTGT12-007	109	1080	1090	10	SCH	192.5	33.2	26.7	79.8	72.9	15.0
OTGT12-007	110	1090	1100	10	SCH	192.5	34.5	27.9	82.4	76.3	15.0
OTGT12-007	111	1100	1110	10	SCH	192.5	32.1	31.9	84.0	78.6	15.0
OTGT12-008	112	1110	1120	10	SCH	102.7	32.6	21.8	65.3	65.4	15.0
OTGT12-008	113	1120	1130	10	SCH	166.3	30.0	20.5	67.8	60.3	15.0
OTGT12-008	114	1130	1140	10	SCH	192.5	29.2	20.6	69.7	59.4	15.0
OTGT12-008	115	1140	1150	10	SCH	192.5	27.7	20.6	68.2	57.6	15.0
OTGT12-008	116	1150	1160	10	SCH	192.5	27.3	20.5	67.8	57.0	15.0
OTGT12-008	117	1160	1170	10	SCH	192.5	32.3	19.1	71.4	61.3	15.0
OTGT12-008	118	1170	1180	10	SCH	192.5	27.5	22.0	69.5	59.2	15.0
OTGT12-008	119	1180	1190	10	SCH	188.7	26.3	20.5	66.4	55.7	15.0
OTGT12-009	120	1190	1200	10	SCH	74.5	34.7	20.5	63.2	66.3	15.0
OTGT12-009	121	1200	1210	10	SCH	137.3	30.9	20.5	65.7	61.5	15.0
OTGT12-009	122	1210	1220	10	SCH	192.5	32.3	26.5	78.8	71.6	15.0
OTGT12-009	123	1220	1230	10	SCH	192.5	30.0	28.4	78.4	71.2	15.0
OTGT12-009	124	1230	1240	10	SCH	192.5	32.5	21.3	73.9	64.7	15.0
OTGT12-010	125	1240	1250	10	SCH	121.7	31.6	20.6	65.0	62.6	15.0
OTGT12-010	126	1250	1260	10	SCH	192.5	37.0	27.5	84.5	78.9	15.0
OTGT12-010	127	1260	1270	10	SCH	192.5	33.3	28.0	81.3	74.8	15.0
OTGT12-010	128	1270	1280	10	SCH	192.5	29.7	22.5	72.2	62.5	15.0
OTGT12-011	129	1280	1290	10	SCH	192.5	31.9	29.2	81.1	74.6	15.0
OTGT12-011	130	1290	1300	10	SCH	192.5	33.6	22.0	75.6	66.8	15.0
OTGT12-012	131	1300	1310	10	SCH	86.3	27.6	22.6	59.5	60.2	15.0
OTGT12-012	132	1310	1320	10	SCH	192.5	31.6	28.8	80.3	73.6	15.0
OTGT12-012	133	1320	1330	10	SCH	192.5	28.1	22.0	70.1	60.0	15.0
OTGT12-012	134	1330	1340	10	SCH	192.5	33.5	20.5	74.0	64.8	15.0
			TOTAL	**1340**							

		IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
	n	134	134	134	134	134	134
SCH	**Average**	135.6	24.1	22.0	60.4	54.9	15.0
	Avg - Stdv	80.8	18.9	18.0	49.6	45.3	15.0
	Avg + Stdv	190.5	29.3	25.9	71.2	64.6	15.0
	Stdv	54.8	5.2	3.9	10.8	9.6	0.0
	CV	0.40	0.22	0.18	0.18	0.17	0.00

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
OT20	1	0	10	10	WAS	34.0	21.0	12.0	37.5	38.7	10.0
OT24	3	10	30	20	WAS	48.6	40.2	22.4	69.7	73.4	20.0
OT24	4	30	40	10	WAS	39.5	18.4	10.1	33.5	33.5	10.0
OT26	5	40	50	10	WAS	25.4	21.6	10.3	35.6	37.4	10.0
OT26	6	50	60	10	WAS	39.5	25.6	28.2	58.8	64.7	10.0
OT27	7	60	70	10	WAS	59.3	22.6	24.8	54.1	56.6	10.0
OT27	8	70	80	10	WAS	45.6	21.9	22.5	49.8	53.0	10.0
OT27	9	80	90	10	WAS	42.9	17.6	22.2	44.8	46.7	10.0
OT31	10	90	100	10	WAS	73.1	17.4	14.9	40.4	37.8	10.0
OT31	11	100	110	10	WAS	39.5	17.3	9.9	32.2	32.0	10.0
OT36	12	110	120	10	WAS	34.6	19.8	11.0	35.3	36.4	10.0
OT36	13	120	130	10	WAS	14.7	23.1	19.6	45.3	50.6	10.0
OT36	14	130	140	10	WAS	18.0	19.6	18.5	41.1	44.9	10.0
OT45	15	140	150	10	WAS	34.5	21.3	17.8	43.6	46.1	10.0
OT45	16	150	160	10	WAS	39.5	19.7	11.7	36.5	36.8	10.0
OT51	17	160	170	10	WAS	39.5	18.0	17.3	40.3	41.6	10.0
OT51	18	170	180	10	WAS	38.2	18.4	14.1	37.4	38.2	10.0
OT51	19	180	190	10	WAS	66.6	23.2	14.9	45.6	45.0	10.0
OT53	20	190	200	10	WAS	78.1	20.8	13.2	42.7	39.9	10.0
OT53	21	200	210	10	WAS	39.5	21.2	12.0	38.2	39.0	10.0
OT53	22	210	220	10	WAS	46.9	21.0	12.0	38.4	38.7	10.0
OT56	23	220	230	10	WAS	13.4	19.5	13.7	35.7	38.9	10.0
OT56	24	230	240	10	WAS	18.0	22.3	9.5	34.7	37.0	10.0
OT56	26	240	260	20	WAS	42.9	47.0	16.4	70.1	73.8	20.0
OT57	27	260	270	10	WAS	45.9	22.7	10.9	39.0	39.4	10.0
OT57	28	270	280	10	WAS	44.2	21.8	7.9	35.0	34.6	10.0
GT1	29	280	290	10	WAS	40.5	15.2	29.1	49.4	51.6	10.0
GT1	30	290	300	10	WAS	18.0	15.0	30.4	48.4	53.0	10.0
GT1	31	300	310	10	WAS	29.5	17.4	27.3	48.7	52.5	10.0
GT2	32	310	320	10	WAS	20.4	15.1	29.4	47.7	51.8	10.0
GT2	33	320	330	10	WAS	18.0	13.7	24.7	41.4	44.5	10.0
GT2	34	330	340	10	WAS	31.0	16.1	21.8	42.1	44.6	10.0
GT2	35	340	350	10	WAS	35.4	12.7	23.3	40.6	41.8	10.0
GT3	36	350	360	10	WAS	18.0	19.2	30.4	52.6	58.7	10.0
GT3	37	360	370	10	WAS	28.1	16.4	30.4	50.7	54.8	10.0
GT3	38	370	380	10	WAS	23.6	17.5	30.4	51.4	56.4	10.0
GT4	39	380	390	10	WAS	23.5	17.6	30.0	51.1	56.0	10.0
GT4	40	390	400	10	WAS	18.0	15.8	30.4	49.2	54.1	10.0
GT4	41	400	410	10	WAS	20.3	14.2	29.9	47.3	51.2	10.0
TC115	42	410	420	10	WAS	29.7	19.3	28.4	51.8	56.4	10.0
TC115	43	420	430	10	WAS	31.0	19.2	21.3	44.7	47.6	10.0
OTM57	44	430	440	10	WAS	18.0	13.9	25.6	42.5	46.0	10.0

Hole_ID	Bench interval	Cumm Length BH		Length (m)	Geot Unit	IRS (Mpa)	FFr	JCr	RMR L90	GSI	mi
		From (m)	To (m)								
TC184	45	440	450	10	WAS	18.0	17.4	29.6	50.1	55.4	10.0
TC184	46	450	460	10	WAS	22.2	18.5	30.4	52.3	57.8	10.0
OT281	47	460	470	10	WAS	18.0	11.2	30.4	44.6	47.8	10.0
OT281	48	470	480	10	WAS	43.6	20.7	26.5	52.6	56.0	10.0
OTGT12-005	49	480	490	10	WAS	44.4	23.7	25.6	54.4	58.8	10.0
OTGT12-005	50	490	500	10	WAS	74.5	26.1	25.6	59.7	62.1	10.0
OTGT12-005	51	500	510	10	WAS	74.5	26.8	22.5	57.3	59.1	10.0
OTGT12-006	52	510	520	10	WAS	59.2	24.6	23.6	54.9	57.6	10.0
OTGT12-006	53	520	530	10	WAS	54.5	22.3	20.5	48.7	50.6	10.0
OTGT12-006	54	530	540	10	WAS	54.5	23.3	23.9	53.2	56.2	10.0
OTGT12-007	55	540	550	10	WAS	60.5	24.5	23.6	54.7	57.4	10.0
OTGT12-007	56	550	560	10	WAS	54.5	25.4	24.5	56.0	59.7	10.0
OTGT12-007	57	560	570	10	WAS	60.3	22.7	24.1	53.4	55.7	10.0
OTGT12-008	58	570	580	10	WAS	54.7	29.7	23.6	59.4	64.1	10.0
OTGT12-008	59	580	590	10	WAS	54.5	23.8	20.5	50.3	52.6	10.0
OTGT12-008	60	590	600	10	WAS	54.5	26.7	20.5	53.2	56.2	10.0
OTGT12-009	61	600	610	10	WAS	60.5	22.0	20.5	49.1	50.3	10.0
OTGT12-009	62	610	620	10	WAS	54.5	21.0	20.5	47.5	49.0	10.0
OTGT12-010	63	620	630	10	WAS	31.6	26.2	23.5	53.6	59.5	10.0
OTGT12-010	64	630	640	10	WAS	46.8	25.8	20.5	51.7	55.2	10.0
OTGT12-010	65	640	650	10	WAS	54.5	26.6	20.5	53.1	56.2	10.0
OTGT12-011	66	650	660	10	WAS	23.7	24.3	20.5	47.8	53.2	10.0
OTGT12-011	67	660	670	10	WAS	25.1	25.3	20.5	49.4	54.5	10.0
OTGT12-011	68	670	680	10	WAS	54.5	28.2	22.0	56.2	60.1	10.0
OTGT12-011	69	680	690	10	WAS	54.5	28.7	20.5	55.2	58.8	10.0
OTGT12-012	70	690	700	10	WAS	54.5	30.0	23.6	59.6	64.5	10.0
OTGT12-012	71	700	710	10	WAS	54.5	25.2	20.5	51.7	54.3	10.0
OTGT12-012	72	710	720	10	WAS	54.5	24.0	20.5	50.5	52.8	10.0
			TOTAL	**720**							

	n	72	72	72	72	72	72
WAS	**Average**	39.4	21.1	20.7	46.6	49.3	10.0
	Avg - Stdv	22.4	16.9	13.9	39.0	40.5	10.0
	Avg + Stdv	56.4	25.3	27.5	54.2	58.2	10.0
	Stdv	17.0	4.2	6.8	7.6	8.9	0.0
	CV	0.43	0.20	0.33	0.16	0.18	0.00

Appendix B: Laboratory Test Results

ROCKLAB REPORT
2012-SRK-CONSULTING-09

CONFIDENTIAL

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RESULTS OF ROCK PROPERTIES TESTS

Sampling Site: Otjikoto Gold, Namibia

BY

DR J. F. CHEN

Submitted to:

SRK CONSULTING (Pty) Ltd.

6 July 2012

CONTENTS

TABLE 1 RESULTS OF UNIAXIAL COMPRESSIVE STRENGTH TESTS

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Sampling Site: Otjikoto Gold, Namibia

05-06-2012

SPECIMEN PARTICULARS					SPECIMEN DIMENSIONS					SPECIMEN TEST RESULTS			
Rocklab Specimen No. 4817-	Sample ID	Depth From.. m	Depth To.. m	Rock Type	Diameter mm	Height mm	Ratio of Height to Diameter	Mass g	Density g/cm³	Failure Load kN	Strength (UCS) MPa	Failure Code	Note
UCS-005-02	OTGT12-005-2	47.04	47.39	Weathered Albitite Schist	62.98	168.7	2.7	1376.2	2.62	282.9	90.8	3B	
UCS-005-06	OTGT12-005-6	65.29	65.56	Biotite Schist	47.57	132.3	2.8	657.3	2.79	307.8	173.2	3B	
UCS-005-07	OTGT12-005-7	68.10	68.54		47.57	131.9	2.8	665.9	2.84	199.5	112.2	3B	
UCS-005-08	OTGT12-005-8	74.21	74.55		47.57	132.8	2.8	660.5	2.80	274.2	154.3	YA	
UCS-005-11	OTGT12-005-11	136.07	136.48	Albitite	47.54	132.8	2.8	655.0	2.78	317.2	178.7	YA	
UCS-005-13	OTGT12-005-13	15.88	159.18	Schist	47.64	132.5	2.8	642.2	2.72	517.7	290.4	YA	
UCS-005-15	OTGT12-005-15	166.15	166.37		47.65	132.6	2.8	649.6	2.75	387.4	217.2	YA	
UCS-005-16	OTGT12-005-16	167.06	16.31		47.75	132.5	2.8	642.4	2.71	546.0	304.9	YA	
UCS-006-03	OTGT12-006-3	55.10	55.28	Hornfels	47.60	128.8	2.7	620.1	2.70	434.3	244.0	2B	
UCS-006-04	OTGT12-006-4	58.23	58.54		47.57	107.7	2.3	516.7	2.70	494.2	278.1	YA	
UCS-006-05	OTGT12-006-5	63.81	64.12		47.49	71.0	1.5	340.7	2.71	575.7	325.0	YB	
UCS-006-07	OTGT12-006-7	77.33	77.56	OTB Marble	47.46	132.4	2.8	638.2	2.72	152.1	86.0	XA	
UCS-006-09	OTGT12-006-9	79.02	79.31		47.54	132.4	2.8	634.8	2.70	142.9	80.5	XA	
UCS-006-10	OTGT12-006-10	80.92	81.22		47.58	130.1	2.7	625.3	2.70	130.6	73.4	XA	
UCS-006-12	OTGT12-006-12	112.61	112.84	FW Marble	47.44	132.3	2.8	635.0	2.72	130.6	73.9	XA	
UCS-006-13	OTGT12-006-13	113.53	113.86		47.32	132.6	2.8	634.8	2.72	183.1	104.1	XA	
UCS-006-15	OTGT12-006-15	115.26	115.44		47.46	131.3	2.8	626.8	2.70	115.8	65.5	XA	
UCS-006-16	OTGT12-006-16	98.86	99.08	Schist	47.53	132.6	2.8	643.5	2.73	350.1	197.3	YA	
UCS-007-03	OTGT12-007-3	106.95	107.17	Albitite	47.58	127.5	2.7	617.7	2.72	261.6	147.1	XA	
UCS-007-04	OTGT12-007-4	107.31	107.65		47.54	132.3	2.8	636.4	2.71	257.8	145.2	XA	
UCS-007-09	OTGT12-007-9	58.01	58.39	Hornfels	63.08	165.4	2.6	1406.6	2.72	1070.7	342.6	YA	
UCS-007-11	OTGT12-007-11	60.11	60.42		47.66	129.2	2.7	625.7	2.71	693.3	388.6	YB	
UCS-007-12	OTGT12-007-12	61.38	61.92		47.67	127.1	2.7	618.15	2.72	585.39	328.0	YB	
UCS-007-16	OTGT12-007-16	62.23	62.51	OTB Marble	47.69	132.8	2.8	657.16	2.77	157.17	88.0	XA	
UCS-007-17	OTGT12-007-17	62.91	63.21		47.72	129.5	2.7	630.8	2.72	152.6	85.3	XB	
UCS-007-18	OTGT12-007-18	65.18	65.50		47.70	132.0	2.8	638.5	2.71	145.6	81.5	XA	
UCS-007-19	OTGT12-007-19	65.86	66.32		47.70	132.2	2.8	640.6	2.71	142.4	79.7	XB	
UCS-007-20	OTGT12-007-20	67.74	68.00		47.69	132.6	2.8	640.6	2.70	191.8	107.3	XA	
UCS-007-23	OTGT12-007-23	113.03	113.43	Schist	47.46	132.6	2.8	647.9	2.76	363.9	205.7	5B	
UCS-007-24	OTGT12-007-24	75.65	76.00		47.60	136.6	2.9	645.5	2.66	505.9	284.3	YA	
UCS-007-25	OTGT12-007-25	80.25	80.60		47.57	132.3	2.8	638.7	2.72	500.3	281.5	YA	
UCS-007-26	OTGT12-007-26	82.65	83.07		47.66	133.1	2.8	645.0	2.72	483.1	270.8	YA	
UCS-007-28	OTGT12-007-28	93.99	94.35	FW Marble	47.45	131.0	2.8	630.9	2.72	140.0	79.2	XB	
UCS-007-30	OTGT12-007-30	94.77	95.07		47.51	130.8	2.8	628.3	2.71	136.6	77.1	XA	
UCS-007-31	OTGT12-007-31	95.07	95.33		47.45	131.5	2.8	630.9	2.71	129.9	73.4	XA	
UCS-007-32	OTGT12-007-32	95.62	95.86		47.49	132.9	2.8	637.4	2.71	122.1	68.9	XB	
UCS-007-33	OTGT12-007-33	95.86	96.27		47.58	130.4	2.7	628.4	2.71	124.5	70.0	XA	
UCS-007-34	OTGT12-007-34	97.63	97.90		47.69	127.9	2.7	618.6	2.71	121.4	68.0	XA	
UCS-007-35	OTGT12-007-35	97.90	98.22		47.59	132.3	2.8	640.0	2.72	119.2	67.0	XA	
UCS-008-03	OTGT12-008-3	27.80	28.36	Weathered Albitite Schist	62.87	168.7	2.7	1186.6	2.27	45.0	14.5	2B	
UCS-008-04	OTGT12-008-4	36.78	37.05		63.14	168.5	2.7	1370.7	2.60	162.0	51.7	3B	
UCS-008-09	OTGT12-008-9	74.47	74.72	Biotite Schist	47.60	131.4	2.8	640.3	2.74	278.6	156.5	4B	
UCS-008-10	OTGT12-008-10	80.66	80.90		47.58	131.6	2.8	649.3	2.78	278.6	156.7	2B	
UCS-008-11	OTGT12-008-11	82.78	83.05		47.46	132.5	2.8	643.5	2.75	380.4	215.0	3B	
UCS-008-12	OTGT12-008-12	85.83	86.02		47.54	132.2	2.8	649.5	2.77	215.4	121.4	3B	
UCS-008-13	OTGT12-008-13	87.21	87.50		47.56	131.8	2.8	641.5	2.74	371.7	209.2	YA	
UCS-008-15	OTGT12-008-15	141.58	141.90	Hornfels	47.61	132.9	2.8	640.7	2.71	449.5	252.5	YA	
UCS-012-06	OTGT12-012-6	202.19	202.39	FW Marble	47.58	105.3	2.2	508.3	2.72	164.2	92.3	XA	
UCS-012-07	OTGT12-012-7	202.77	202.99		47.58	112.8	2.4	542.6	2.71	137.8	77.5	XA	
UCS-006-17	OTGT12-006-17	4.76	5.09	Calcrete	63.11	162.8	2.6	1293.3	2.54	365.4	116.8	XB	
UCS-006-18	OTGT12-006-18	5.09	5.41		63.20	168.3	2.7	1341.3	2.54	254.6	81.2	XA	
UCS-007-38	OTGT12-007-38	5.60	5.93	Calcrete	63.08	165.1	2.6	1285.0	2.49	308.1	98.6	XA	
UCS-007-39	OTGT12-007-39	7.52	8.46		63.03	161.1	2.6	1248.5	2.48	180.4	57.8	XA	
UCS-008-16	OTGT12-008-16	8.06	8.55	Calcrete	63.03	159.8	2.5	1221.1	2.45	173.9	55.7	XA	
UCS-008-17	OTGT12-008-17	5.55	5.73		62.68	101.4	1.6	789.3	2.52	88.0	28.5	XB	
UCS-009-05	OTGT12-009-5	1.79	2.13	Calcrete	63.14	165.4	2.6	1201.7	2.32	112.9	36.1	XB	
UCS-009-06	OTGT12-009-6	3.80	4.06		63.20	170.6	2.7	1206.3	2.25	39.2	12.5	XA	

Note: All tests were conducted according to the ISRM's (International Society for Rock Mechanics) specification.

TABLE 2 RESULTS OF UNIAXIAL COMPRESSION TESTS WITH ELASTIC MODULUS AND POISSON RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES

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13-06-2012

SPECIMEN PARTICULARS					SPECIMEN DIMENSIONS					SPECIMEN TEST RESULTS								
Rocklab Specimen No 4817-	Sample ID	Depth From m	Depth To m	Rock Type	Diameter mm	Height mm	Ratio of Height to diameter	Mass g	Density g/cm³	Failure Load kN	Strength (UCS) MPa	Tangent Elastic Modulus @ 50% UCS GPa	Secant Elastic Modulus @ 50% UCS GPa	Poisson's Ratio Tangent @ 50% UCS	Poisson's Ratio Secant @ 50% UCS	Linear Axial Strain at Failure mm/mm	Failure Code	Note
UCM-05-10A					47.57	95.5	2.0	468.0	2.76	352.1	198.1	61.8	72.9	0.728	0.334	0.003516	XA	
UCM-05-10B	OTGT12-005-10	133.50	134.88	Albitite	47.56	99.5	2.1	486.7	2.75	343.8	193.5	56.3	79.3	0.506	0.305	0.003566	XB	
UCM-05-12G	OTGT12-005-12	163.47	164.46	Schist	47.63	97.0	2.0	474.3	2.74	428.0	240.2	74.2	78.1	0.346	0.262	0.003688	YA	
UCM-07-21	OTGT12-007-21	118.47	119.86	Schist	47.48	100.0	2.1	489.3	2.76	342.4	193.4	58.5	57.2	0.208	0.172	0.003565	YA	
UCM-09-01	OTGT-12-009-1	80.89	81.77	Albitite	47.42	98.9	2.1	475.4	2.72	165.6	93.8	45.3	42.2	0.225	0.160	0.002096	4B	
UCM-09-03	OTGT-12-009-3	36.31	37.76	Weathered Albitite Schist	62.88	129.8	2.1	1032.7	2.56	161.3	51.9	11.6	17.6	0.162	0.114	0.004529	3B	
UCM-12-02	OTGT-12-012-2	164.16	164.41	OTB Marble	47.50	134.4	2.8	649.0	2.73	147.3	83.1	53.2	52.1	0.340	0.245	0.002358	XB	
UCM-12-04	OTGT-12-012-4	164.95	165.45	OTB Marble	47.47	130.4	2.7	627.2	2.72	189.3	107.0	56.7	52.3	0.320	0.210	0.003062	XA	
UCM-12-08	OTGT-12-012-8	203.93	205.35	FW Marble	47.58	96.6	2.0	465.6	2.71	153.3	86.2	54.6	41.8	0.372	0.179	0.002113	XA	
UCM-12-09	OTGT-12-012-9	208.22	208.57	FW Marble	47.59	128.2	2.7	614.7	2.70	136.1	76.5	51.8	38.7	0.305	0.152	0.002275	XA	
UCM-12-10	OTGT-12-012-10	209.66	210.14	FW Marble	47.62	127.8	2.7	616.0	2.71	149.2	83.8	51.7	43.5	0.341	0.191	0.002387	XB	

Note: All tests were conducted according to the ISRM's Specification.

TABLE 3 RESULTS OF TRIAXIAL COMPRESSIVE STRENGTH TESTS

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Sampling Site: **Otjikoto** Gold, Namibia

13-Jun-12

SPECIMEN PARTICULARS				SPECIMEN DIMENSIONS					SPECIMEN TEST RESULTS				
Rocklab Specimen No 4817-	Sample ID	Depth From.. To.. m	Rock Type	Diameter mm	Height mm	Ratio of Height to Diameter	Mass g	Density g/cm³	Confining Pressure δ_3 MPa	Failure Load P kN	Strength (TCS) δ_1 MPa	Failure Mode	Note
TCS-005-01A	OTGT12-005-1	31.18 - 32.64		62.90	124.6	2.0	1015.2	2.62	10.0	600.0	193.1	XA	
TCS-005-01B				62.81	123.9	2.0	1012.9	2.64	20.0	737.8	238.1	2B	
TCS-005-01C				63.03	122.9	2.0	1005.2	2.62	30.0	1093.0	350.3	5B	
TCS-005-01D				62.89	122.4	1.9	998.4	2.63	10.0	541.5	174.3	XA	
TCS-005-01E				62.91	124.3	2.0	1016.7	2.63	20.0	701.7	225.7	4B	
TCS-005-01F				62.94	123.2	2.0	1007.5	2.63	30.0	1007.6	323.9	2B	
TCS-005-04A	OTGT12-005-4	55.71 - 56.55		47.55	100.0	2.1	497.5	2.80	10.0	337.3	189.9	3B	
TCS-005-04B				47.55	101.3	2.1	503.7	2.80	20.0	454.5	256.0	3B	
TCS-005-04C				47.54	99.1	2.1	492.8	2.80	30.0	521.5	293.8	XA	
TCS-005-04D				47.56	99.7	2.1	495.2	2.80	10.0	290.2	163.4	3B	
TCS-005-04E				47.54	101.0	2.1	502.6	2.80	20.0	382.6	215.6	3B	
TCS-005-04F				47.55	97.9	2.1	486.9	2.80	30.0	489.5	275.6	3B	
TCS-005-10A	OTGT12-005-10	133.50 - 134.88		47.61	99.3	2.1	487.9	2.76	10.0	555.3	311.9	XA	
TCS-005-10B				47.60	100.0	2.1	488.0	2.74	20.0	672.5	377.9	XA	
TCS-005-10C				47.60	99.4	2.1	485.4	2.74	30.0	781.1	438.9	3B	
TCS-005-10D				47.62	99.7	2.1	489.2	2.76	10.0	502.8	282.3	XA	
TCS-005-10E				47.68	95.9	2.0	470.1	2.74	20.0	688.5	385.6	XA	
TCS-005-10F				47.58	94.0	2.0	459.8	2.75	30.0	772.6	434.5	3B	
TCS-005-12A	OTGT12-005-12	163.47 - 164.46		47.63	100.3	2.1	491.4	2.75	10.0	525.5	294.9	3B	
TCS-005-12B				47.63	100.5	2.1	490.7	2.74	20.0	648.0	363.7	XA	
TCS-005-12C				47.65	100.8	2.1	492.7	2.74	30.0	769.3	431.4	XA	
TCS-005-12D				47.65	99.3	2.1	486.0	2.74	10.0	511.6	286.9	3B	
TCS-005-12E				47.64	98.4	2.1	481.1	2.74	20.0	637.1	357.4	XA	
TCS-005-12F				47.64	98.5	2.1	481.5	2.74	30.0	682.8	383.1	3B	
TCS-006-01A	OTGT12-006-1	59.93 - 60.94		47.52	95.8	2.0	461.7	2.72	10.0	688.0	387.9	XA	
TCS-006-01B				47.52	95.1	2.0	459.6	2.72	20.0	788.3	444.5	XA	
TCS-006-01C				47.51	95.7	2.0	460.9	2.72	30.0	1034.8	583.7	XA	
TCS-006-01D				47.51	101.1	2.1	488.9	2.73	10.0	749.2	422.6	XA	
TCS-006-01E				47.52	100.3	2.1	483.7	2.72	20.0	904.9	510.2	XA	
TCS-006-01F				47.52	96.3	2.0	462.6	2.71	30.0	1068.0	602.2	XA	
TCS-006-06A	OTGT12-006-6	75.27 - 76.64		47.56	100.8	2.1	488.3	2.73	10.0	317.0	178.5	XA	
TCS-006-06B				47.59	96.3	2.0	464.7	2.71	20.0	518.0	291.2	XA	
TCS-006-06C				47.55	96.2	2.0	464.5	2.72	30.0	526.1	296.3	7B	
TCS-006-06D				47.53	95.0	2.0	456.5	2.71	10.0	244.2	137.6	XA	
TCS-006-06E				47.55	101.1	2.1	488.8	2.72	20.0	363.8	204.9	7B	
TCS-006-06F				47.55	99.5	2.1	479.4	2.71	30.0	420.4	236.7	7B	
TCS-006-11A	OTGT12-006-11	110.86 - 111.88		47.47	91.2	1.9	437.5	2.71	10.0	266.0	150.3	XA	
TCS-006-11B				47.49	91.1	1.9	436.6	2.71	20.0	326.8	184.5	XA	
TCS-006-11C				47.53	94.0	2.0	450.9	2.70	30.0	413.9	233.3	XA	
TCS-006-11D				47.48	94.3	2.0	453.0	2.71	10.0	214.2	121.0	XA	
TCS-006-11E				47.40	91.4	1.9	438.3	2.72	20.0	327.0	185.3	XA	
TCS-006-11F				47.39	95.2	2.0	455.6	2.71	30.0	398.6	226.0	XA	
TCS-007-01A	OTGT12-007-1	104.86 - 106.19		47.57	96.1	2.0	465.4	2.73	10.0	389.7	219.3	XA	
TCS-007-01B				47.52	96.3	2.0	466.0	2.73	20.0	461.4	260.2	XA	
TCS-007-01C				47.57	101.8	2.1	492.7	2.72	30.0	1027.9	578.3	XA	
TCS-007-01D				47.55	94.1	2.0	454.6	2.72	10.0	390.2	219.7	XA	
TCS-007-01E				47.51	96.2	2.0	463.8	2.72	20.0	458.1	258.4	XA	
TCS-007-01F				47.52	93.7	2.0	452.4	2.72	30.0	468.3	264.1	4B	
TCS-007-07A	OTGT12-007-7	56.01 - 57.47		63.06	127.4	2.0	1082.6	2.72	10.0	1297.7	415.5	XA	
TCS-007-07B				63.11	127.4	2.0	1083.0	2.72	20.0	1545.3	494.0	XA	
TCS-007-07C				47.58	98.9	2.1	476.7	2.71	20.0	941.6	529.6	XA	
TCS-007-07D				47.56	90.7	1.9	432.9	2.69	30.0	1117.1	628.8	XA	
TCS-007-13A				47.63	100.2	2.1	483.8	2.71	10.0	242.4	136.0	XA	
TCS-007-13B				47.63	100.6	2.1	485.8	2.71	20.0	341.6	191.7	XA	

TCS-007-13C	OTGT12-007-13	63.95 - 64.87		47.66	99.9	2.1	484.2	2.72	30.0	423.3	237.3	XA	
TCS-007-13D				47.67	100.1	2.1	486.7	2.73	10.0	245.1	137.3	XA	
TCS-007-13E				47.67	100.1	2.1	501.0	2.80	20.0	316.9	177.5	XA	
TCS-007-13F				47.67	101.0	2.1	494.2	2.74	30.0	420.4	235.5	XA	
TCS-007-14A	OTGT12-007-14	66.72 - 67.50		47.42	94.8	2.0	452.8	2.70	10.0	214.0	121.2	XA	
TCS-007-14B				47.39	95.6	2.0	456.2	2.71	20.0	322.7	183.0	XA	
TCS-007-14C				47.70	96.2	2.0	464.0	2.70	30.0	407.5	228.0	XA	
TCS-007-14D				47.67	94.9	2.0	457.9	2.70	10.0	230.4	129.1	XA	
TCS-007-14E				47.67	101.0	2.1	487.3	2.70	20.0	335.9	188.2	XA	
TCS-007-14F				47.69	95.4	2.0	461.3	2.71	30.0	424.5	237.6	XA	
TCS-007-21A	OTGT12-007-21	118.47 - 119.86		47.52	97.3	2.0	474.5	2.75	10.0	381.7	215.2	3B	
TCS-007-21B				47.48	96.2	2.0	470.3	2.76	20.0	522.3	295.0	XA	
TCS-007-21C				47.47	96.4	2.0	473.7	2.78	30.0	620.1	350.4	XA	
TCS-007-21D				47.48	95.6	2.0	467.5	2.76	10.0	308.6	174.3	3B	
TCS-007-21E				47.48	101.8	2.1	494.3	2.74	20.0	521.5	294.6	XA	
TCS-007-21F				47.52	95.1	2.0	463.2	2.75	30.0	656.3	370.0	XA	
TCS-007-27A	OTGT12-007-27	92.86 - 93.99		47.50	100.9	2.1	487.1	2.72	10.0	253.9	143.3	XA	
TCS-007-27B				47.62	86.9	1.8	420.2	2.71	20.0	352.0	197.6	XA	
TCS-007-27C				47.56	99.9	2.1	483.0	2.72	30.0	420.5	236.7	XA	
TCS-007-27D				47.62	81.8	1.7	434.1	2.98	10.0	259.2	145.6	XA	
TCS-007-27E				47.63	99.2	2.1	478.1	2.70	20.0	349.7	196.2	XA	
TCS-007-27F				47.53	100.0	2.1	481.5	2.71	30.0	426.3	240.3	XA	
TCS-007-29A	OTGT12-007-29	99.44 - 100.38		47.57	99.9	2.1	482.3	2.72	10.0	259.8	146.2	XA	
TCS-007-29B				47.57	92.0	1.9	443.6	2.71	20.0	343.2	193.1	XA	
TCS-007-29C				47.63	94.0	2.0	453.6	2.71	30.0	423.6	237.7	XA	
TCS-007-29D				47.59	96.2	2.0	462.9	2.70	10.0	311.1	174.9	XA	
TCS-007-29E				47.62	95.3	2.0	458.3	2.70	20.0	384.8	216.1	XA	
TCS-007-29F				47.59	94.9	2.0	456.6	2.70	30.0	398.7	224.2	XA	
TCS-008-01A	OTGT12-008-1	37.99 - 38.91		63.23	124.7	2.0	1017.1	2.60	10.0	403.8	128.6	4B	
TCS-008-01B				63.10	125.3	2.0	1022.0	2.61	20.0	556.2	177.9	4B	
TCS-008-01C				63.21	121.1	1.9	977.8	2.57	30.0	689.9	219.8	4B	
TCS-008-01D				63.15	124.0	2.0	1004.2	2.59	10.0	338.8	108.2	3B	
TCS-008-01E				63.16	124.4	2.0	1004.5	2.58	20.0	452.5	144.4	5B	
TCS-008-08A	OTGT12-008-8	96.89 - 97.80		47.58	100.9	2.1	489.2	2.73	10.0	484.1	272.3	XA	
TCS-008-08B				47.57	100.1	2.1	487.0	2.74	20.0	582.9	327.9	XA	
TCS-008-08C				47.57	98.0	2.1	478.4	2.75	30.0	663.3	373.2	3B	
TCS-008-08D				47.58	98.1	2.1	476.6	2.73	10.0	485.0	272.8	XA	
TCS-008-08E				47.57	100.2	2.1	487.1	2.74	20.0	585.3	329.3	XA	
TCS-008-08F				47.57	99.1	2.1	481.8	2.74	30.0	614.0	345.4	3B	
TCS-09-01A	OTGT12-009-1	80.89	81.77	47.56	96.3	2.0	474.6	2.77	10.0	497.9	280.3	3B	
TCS-09-01B				47.53	97.9	2.1	476.6	2.74	20.0	655.1	369.2	4B	
TCS-09-01C				47.57	96.4	2.0	453.7	2.65	30.0	316.0	177.8	4B	
TCS-09-03A	OTGT12-009-3	36.31	37.76	62.27	130.7	2.1	1012.6	2.54	10.0	258.9	85.0	2B	
TCS-09-03B				62.71	128.4	2.0	1031.9	2.60	20.0	434.2	140.6	3B	
TCS-09-03C				62.74	130.3	2.1	1058.2	2.63	30.0	646.6	209.2	3B	
TCS-012-01A	OTGT12-012-1	162.76	163.98	47.54	95.9	2.0	460.7	2.71	10.0	226.6	127.6	XA	
TCS-012-01B				47.55	96.3	2.0	462.8	2.71	20.0	302.3	170.2	XA	
TCS-012-01C				47.53	95.6	2.0	460.8	2.72	30.0	426.9	240.6	XA	
TCS-012-03A	OTGT12-012-3	167.66	169.10	47.44	94.2	2.0	450.8	2.71	10.0	271.0	153.3	XA	
TCS-012-03B				47.53	93.5	2.0	448.4	2.70	20.0	352.0	198.4	XA	
TCS-012-03C				47.46	93.8	2.0	450.6	2.72	30.0	456.7	258.1	XA	
TCS-012-05E	OTGT12-012-5	200.66	201.98	47.58	94.3	2.0	453.8	2.71	10.0	242.4	136.3	7B	
TCS-012-05F				47.57	94.0	2.0	452.5	2.71	20.0	322.9	181.7	XA	
TCS-012-05G				47.57	93.85	2.0	452.10	2.71	30.00	377.43	212.4	XA	

Note: All tests were conducted according to the ISRM's Specification.

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TABLE 4 RESULTS OF DIRECT TENSILE STRENGTH TESTS

Client: SRK Consulting

Sampling Site: Otjikoto Gold, Namibia

08-06-2012

SPECIMEN PARTICULARS				SPECIMEN DIMENSIONS						
ROCKLAB Specimen No 4817-	Sample ID	Sample depth From.. To.. m	Rock Type	Diameter mm	Height mm	Mass gram	Density g/cm³	Failure Load kN	Direct Tensile Strength MPa	Note
DT-005-09	OTGT12-005-9	122.22 - 122.81		47.59	101.35	507.4	2.81	16.0	9.0	
DT-005-14	OTGT12-005-14	161.88 - 162.48		47.61	113.49	556.7	2.76	22.9	12.9	
DT-007-02	OTGT12-007-2	107.86 - 108.22		47.58	102.66	502.5	2.75	9.9	5.6	
DT-007-05	OTGT12-007-5	42.19 - 42.70		48.75	128.06	1198.7	5.01	14.5	7.8	
DT-007-06	OTGT12-007-6	54.72 - 55.21		47.64	130.78	1113.5	4.78	22.6	12.7	
DT-007-22	OTGT12-007-22	110.39 - 110.86		47.49	99.07	488.9	2.79	13.1	7.4	
DT-008-06	OTGT12-008-6	66.34 - 67.02		47.59	99.65	484.5	2.73	23.4	13.2	
DT-008-07	OTGT12-008-7	72.16 - 72.69		47.57	111.44	492.2	2.49	10.4	5.9	
DT-008-14	OTGT12-008-14	139.44 - 140.00		47.58	101.28	488.3	2.71	9.2	5.2	
DT-009-02	OTGT12-009-2	69.66 - 69.87	Schist	47.47	111.90	539.4	2.72	41.2	23.3	
DT-010-01	OTGT12-010-1	119.93 - 120.53	Albitite	47.48	110.75	534.0	2.72	45.3	25.6	

Note: All tests were conducted according to the ISRM's suggested method.

TABLE 5 RESULTS OF BASE FRICTION ANGLE TESTS OF ROCKS BASED ON DIRECT SHEAR TESTS ON SAW-CUT ROCK SURFACE

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Client: SRK Consulting

Project : Otjikoto Gold, Namibia

30-Jun-12

ROCKLAB Specimen No 4817-	Sample ID	Depth of Core From .. To (m)	Rock Type	Shear Area (mm²)	Shear Cycle No	Hor. Force (kN)	Vert. Force (kN)	Dil. Angle (°)	Normal Stress (MPa)	Shear Stress (MPa)	Angle or App. Angle (°)	Base Friction Angle (°)	Note
BFA-005-3	OTGT12-005-3	53.42 - 53.71		1554	1	0.83	1.00	0.0	0.64	0.54	39.8		
				1450	2	1.48	2.00	0.0	1.38	1.02	36.4	37.0	
				1370	3	2.25	3.00	0.0	2.19	1.65	36.9		
BFA-005-5	OTGT12-005-5	58.29 - 58.45		1602	1	0.86	1.00	0.0	0.63	0.54	40.8		
				1600	2	1.52	2.00	0.0	1.25	0.95	37.3	36.0	
				1592	3	2.08	3.00	0.0	1.89	1.30	34.7		
BFA-006-2	OTGT12-006-2	53.92 - 54.09		1598	1	0.79	1.00	0.0	0.63	0.49	38.2		
				1466	2	1.57	2.00	0.0	1.36	1.07	38.1	39.0	
				1411	3	2.43	3.00	0.0	2.13	1.72	39.0		
BFA-006-8	OTGT12-006-8	77.89 - 78.01		1572	1	1.04	1.00	0.0	0.64	0.67	46.3		
				1524	2	1.73	2.00	0.0	1.31	1.13	40.8	40.0	
				1428	3	2.47	3.00	0.0	2.10	1.73	39.5		
BFA-006-14	OTGT12-006-14	115.12 - 115.26											1
BFA-007-8	OTGT12-007-8	57.79 - 58.01		3007	1	0.79	1.00	0.0	0.33	0.26	38.4		
				2862	2	1.30	2.00	0.0	0.70	0.45	33.0	34.0	
				2868	3	2.01	3.00	0.0	1.05	0.70	33.8		
BFA-007-10	OTGT12-007-10	58.80 - 59.01		2903	1	0.89	1.00	0.0	0.34	0.31	41.6		
				2730	2	1.72	2.00	0.0	0.73	0.63	40.7	40.0	
				2622	3	2.46	3.00	0.0	1.14	0.94	39.3		
BFA-007-15	OTGT12-007-15	62.68 - 62.86		1596	1	1.04	1.00	0.0	0.63	0.65	46.1		
				1583	2	1.86	2.00	0.0	1.26	1.18	43.0	42.0	
				1577	3	2.56	3.00	0.0	1.90	1.62	40.4		
BFA-007-36	OTGT12-007-36	98.22 - 98.41		1647	1	0.95	1.00	0.0	0.61	0.58	43.5		
				1555	2	1.79	2.00	0.0	1.29	1.15	41.9	41.0	
				1723	3	2.56	3.00	0.0	1.74	1.49	40.5		
BFA-007-37	OTGT12-007-37	101.65 - 101.86		1606	1	0.58	1.00	0.0	0.62	0.36	30.1		
				1518	2	1.22	2.00	0.0	1.32	0.80	31.3	32.0	
				1481	3	1.94	3.00	0.0	2.03	1.31	32.8		
BFA-008-2	OTGT12-008-2	27.33 - 27.53		2920	1	0.76	1.00	0.0	0.34	0.26	37.1		
				2901	2	1.36	2.00	0.0	0.69	0.47	34.2	33.0	
				2906	3	1.93	3.00	0.0	1.03	0.66	32.7		
BFA-008-5	OTGT12-008-5	37.66 - 37.85		2930	1	0.85	1.00	0.0	0.34	0.29	40.3		
				2894	2	1.47	2.00	0.0	0.69	0.51	36.2	36.0	
				2874	3	2.14	3.00	0.0	1.04	0.74	35.5		
BFA-009-04	OTGT12-009-4	38.20 - 38.63											1

Note: The tests were conducted using ROCKLAB servo-controlled direct rock shear testing equipment.
They were conducted according to the ASTM standard D5607-95.

1 - No suiable specimen could be prepared for the test.

APPENDIX 1

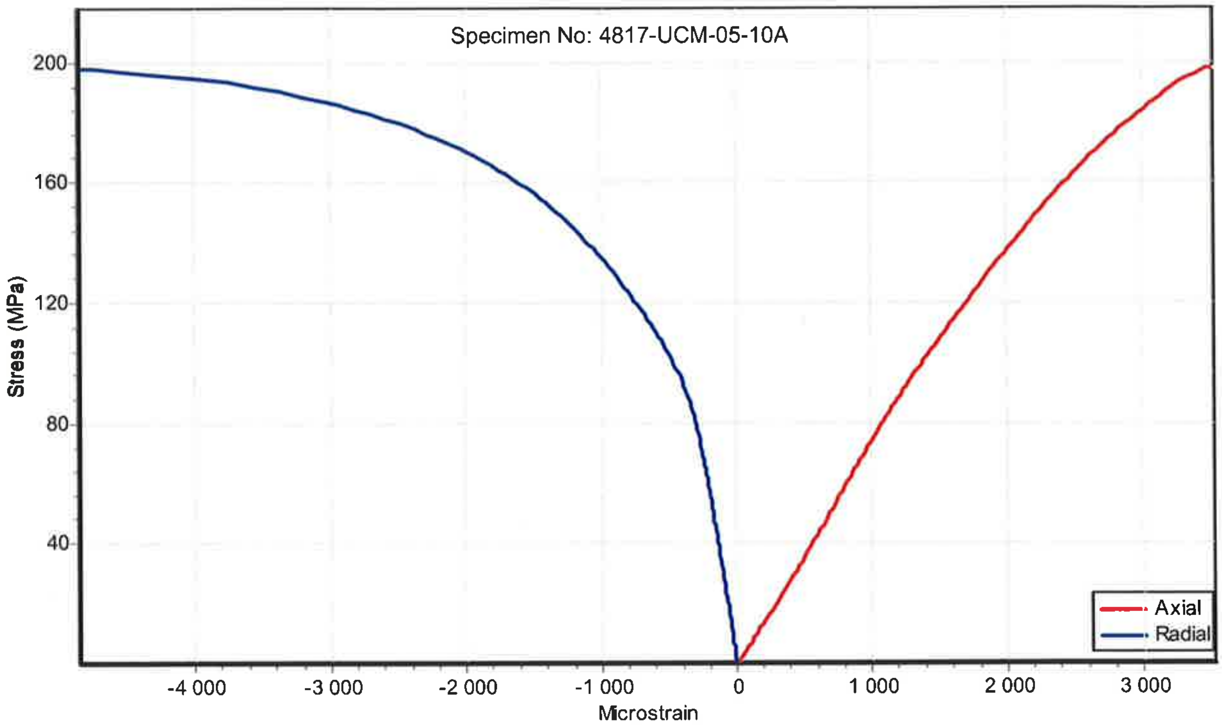
STRESS VIA STRAIN CURVES FOR UCM TESTS

UNIACIAL COMPRESSION TEST

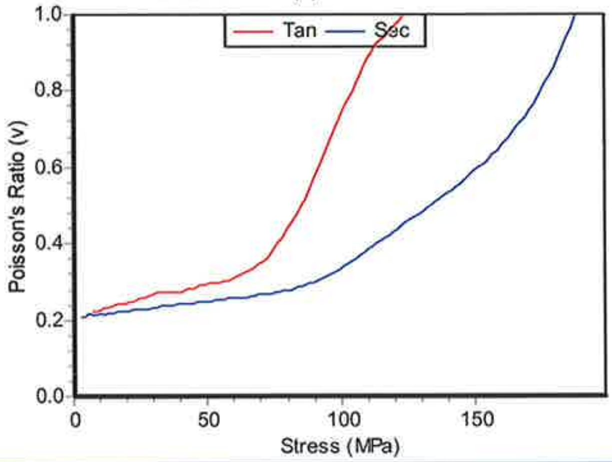
2012/06/13 08:45:41 AM

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES

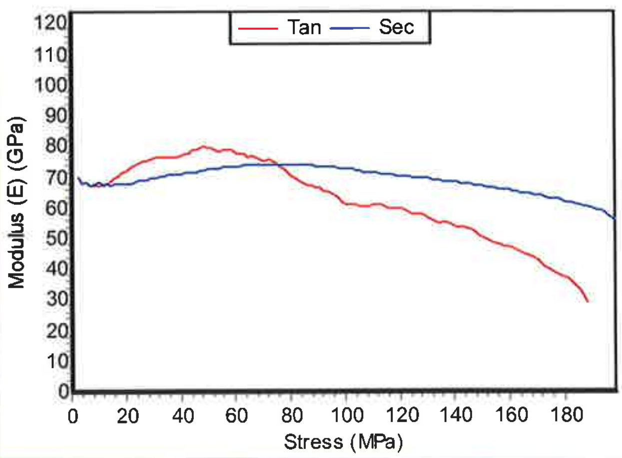
Stress / Strain Curve



Poisson's Ratio (v) / Stress Curve



Modulus (E) / Stress Curve



Failure Load: 352.1 kN | Peak Strength: 198.1 MPa | Axial Strain at Failure: 3516 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	19.8	72.4	67.7	0.247	0.225
20	39.6	76.9	71.2	0.273	0.242
30	59.4	77.8	73.6	0.314	0.256
40	79.2	71.6	74.2	0.434	0.277
50	99	61.8	72.9	0.728	0.334
60	119	60	70.6	0.964	0.430
70	139	54.6	68.3	1.160	0.534
80	158	47.3	65.7	1.622	0.656
90	178	37.7	62.4	2.694	0.860

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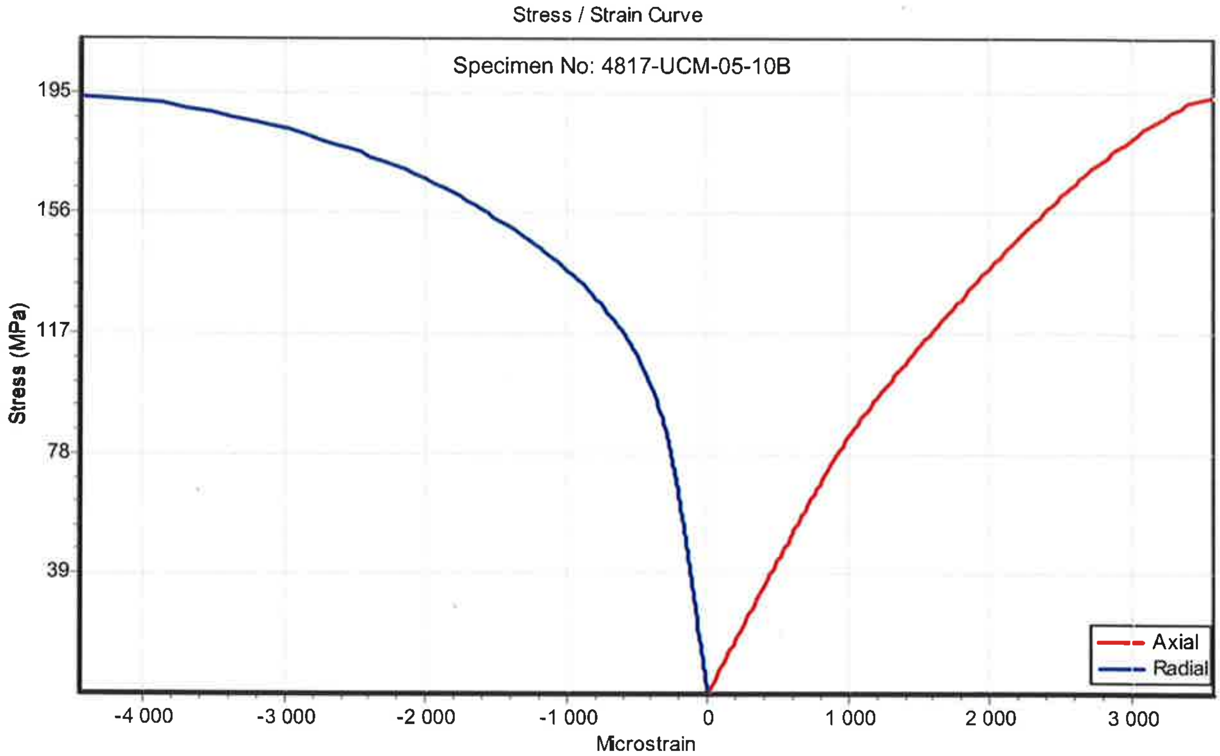
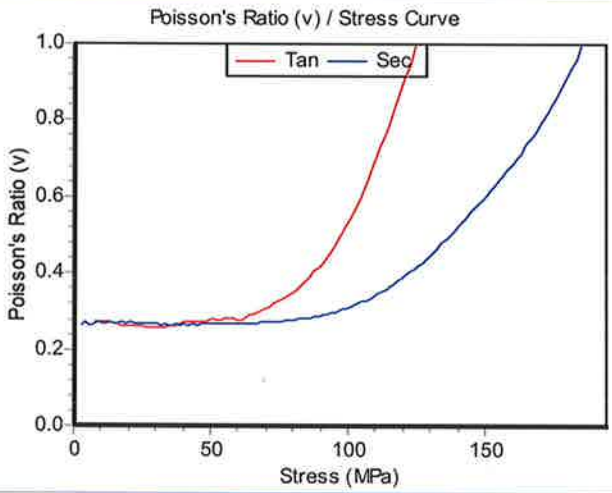
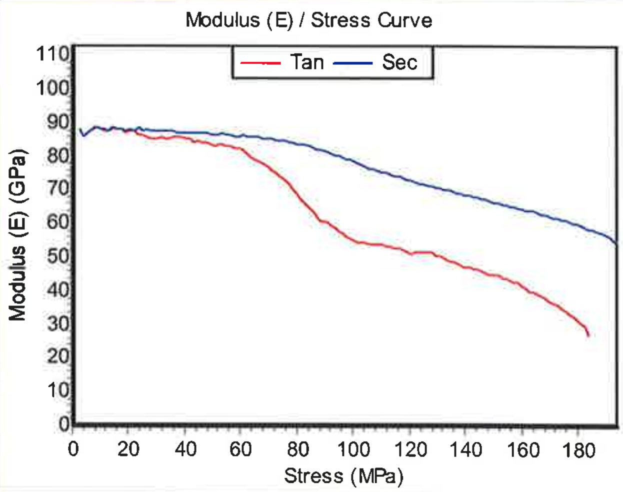
UNIAXIAL COMPRESSION TEST

2012/06/13 08:47:05 AM

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES







Failure Load: 343.8 kN | Peak Strength: 193.5 MPa | Axial Strain at Failure: 3566 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	19.4	87.4	88.2	0.262	0.271
20	38.7	85.9	87.4	0.269	0.263
30	58.1	82.8	86.2	0.281	0.269
40	77.4	70.4	84.2	0.347	0.277
50	96.8	56.3	79.3	0.506	0.305
60	116	52.6	74	0.820	0.372
70	135	48.8	69.7	1.243	0.488
80	155	43.1	65.5	1.606	0.653
90	174	35.1	61.4	2.240	0.840

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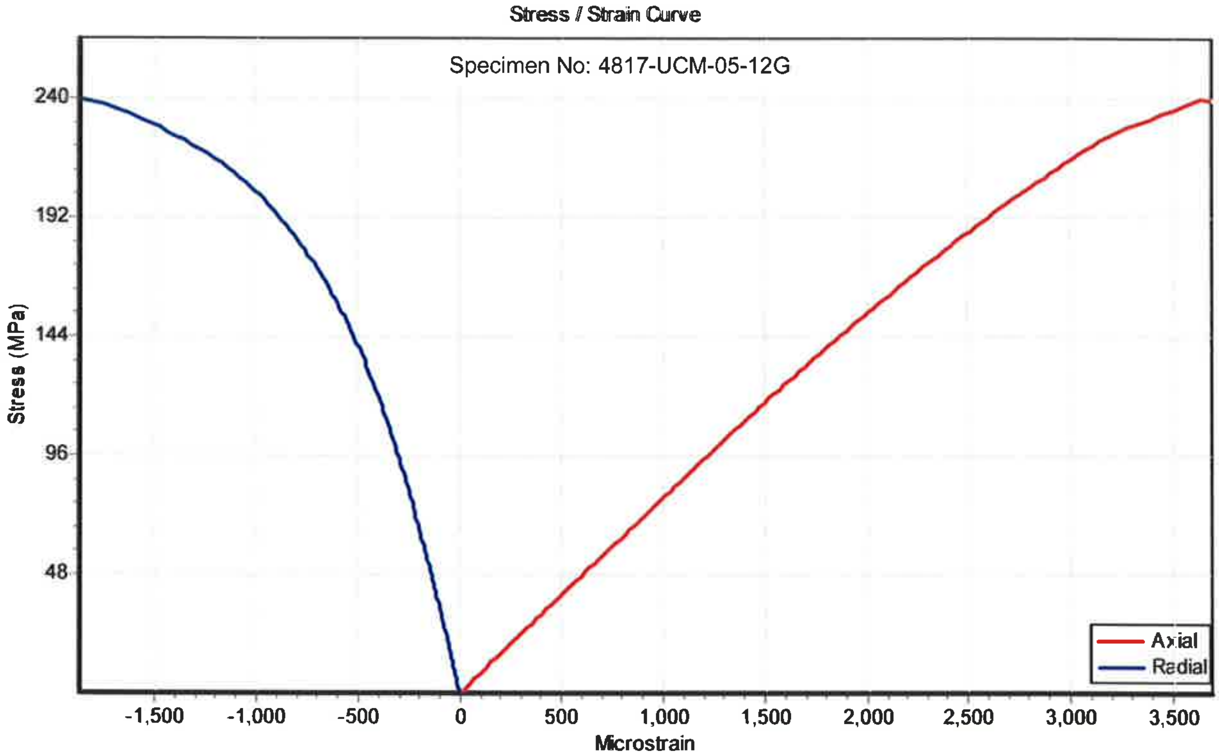
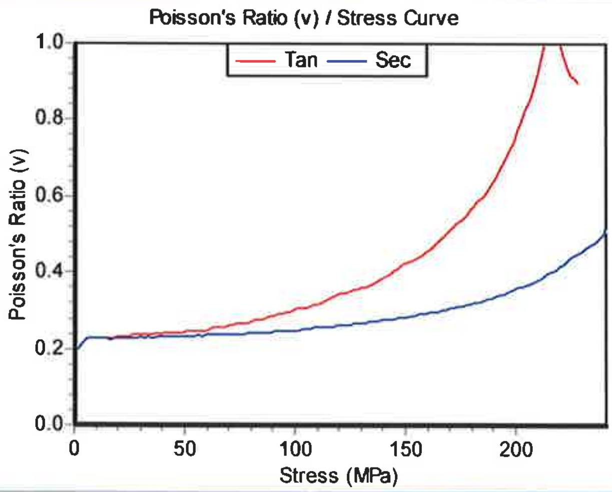
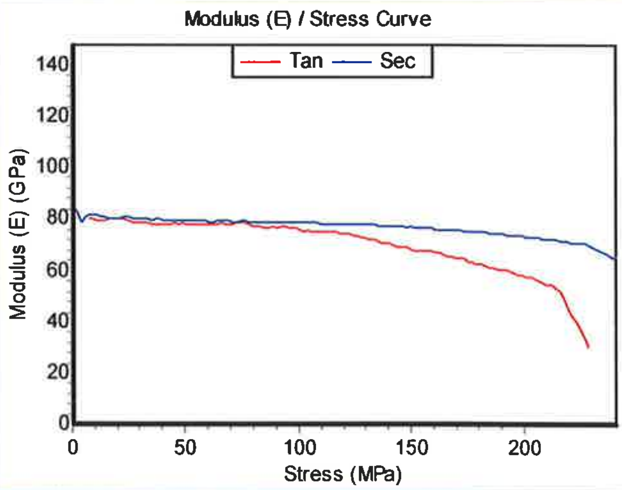
UNIAXIAL COMPRESSION TEST

6/12/2012 12:21:04 PM

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES







Failure Load: 428 kN | Peak Strength: 240.2 MPa | Axial Strain at Failure: 3688 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	24	79.3	80.7	0.233	0.227
20	48	78.5	79.6	0.245	0.234
30	72.1	78.5	78.9	0.266	0.240
40	96.1	76.5	78.9	0.299	0.249
50	120	74.2	78.1	0.346	0.262
60	144	69.4	77.1	0.407	0.279
70	168	65	75.6	0.511	0.306
80	192	60.1	73.9	0.664	0.340
90	216	49.7	71.5	1.050	0.407

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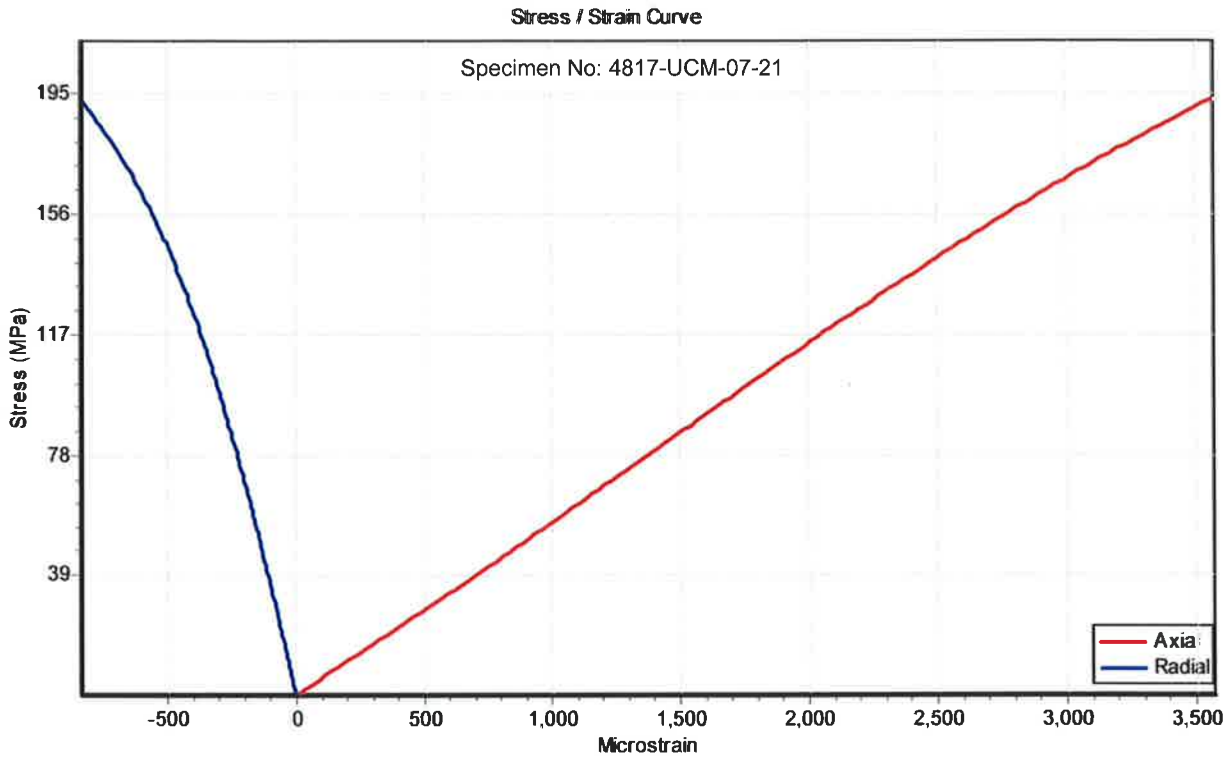
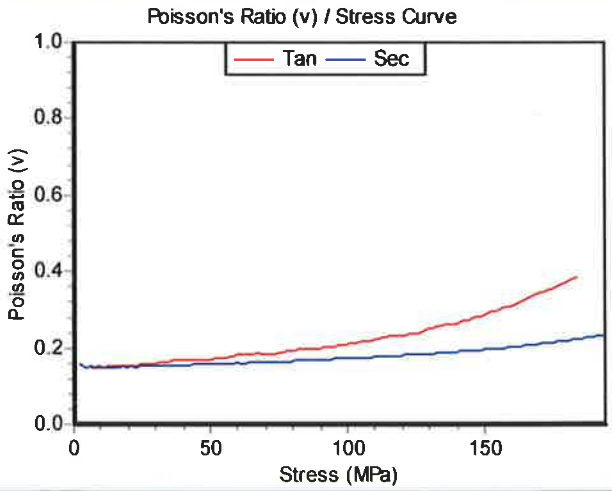
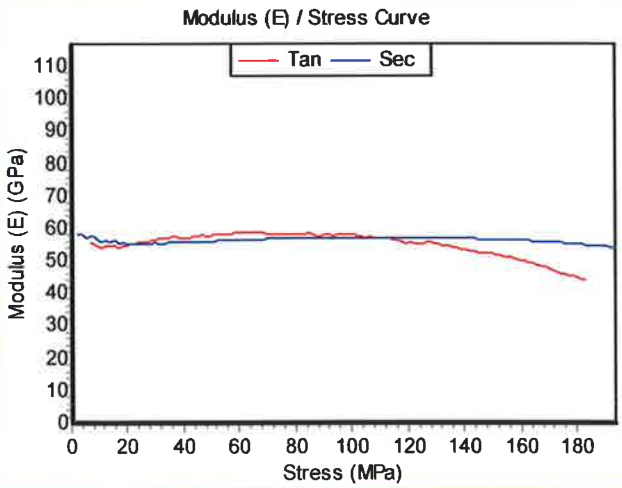
UNIACIAL COMPRESSION TEST

6/12/2012 12:21:22 PM

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES







Failure Load: 342.4 kN | Peak Strength: 193.4 MPa | Axial Strain at Failure: 3565 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	19.3	54.5	55.3	0.154	0.150
20	38.7	57.3	55.8	0.168	0.154
30	58	58.5	56.5	0.179	0.160
40	77.4	58	57	0.192	0.165
50	96.7	58.5	57.2	0.208	0.172
60	116	56.3	57.2	0.231	0.179
70	135	54.1	56.9	0.261	0.189
80	155	51.4	56.5	0.302	0.200
90	174	45.7	55.6	0.356	0.214

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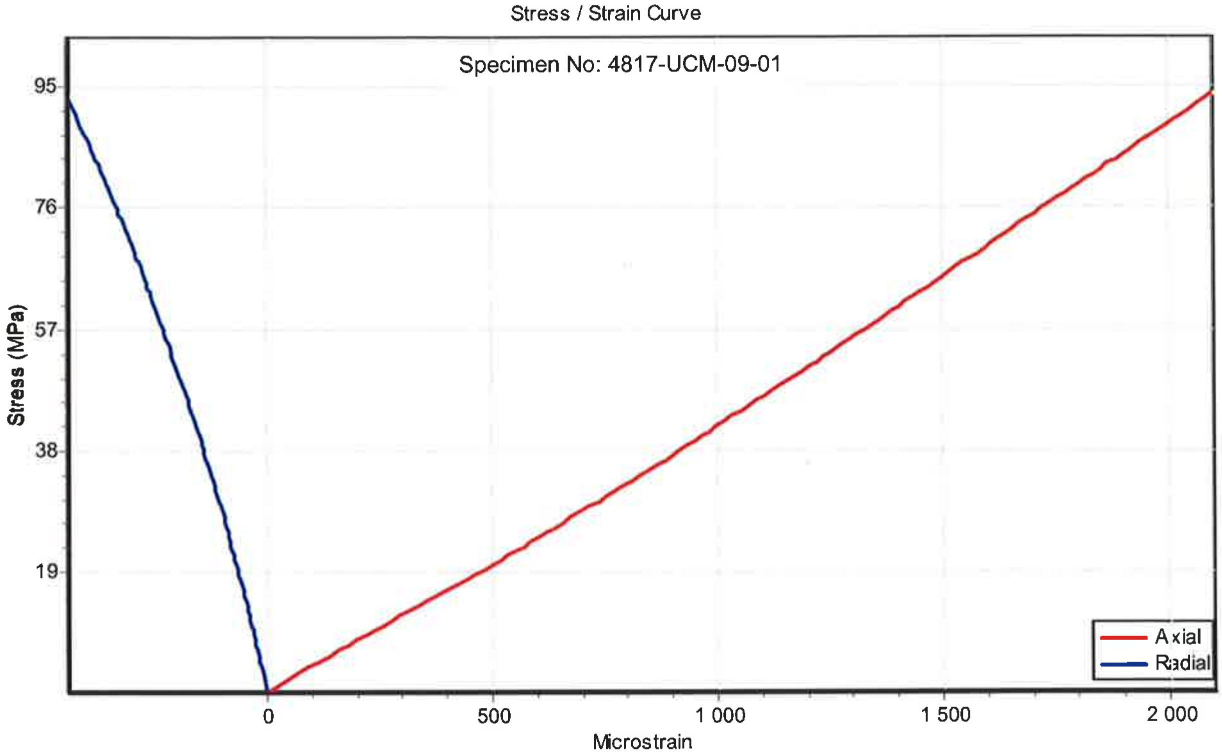
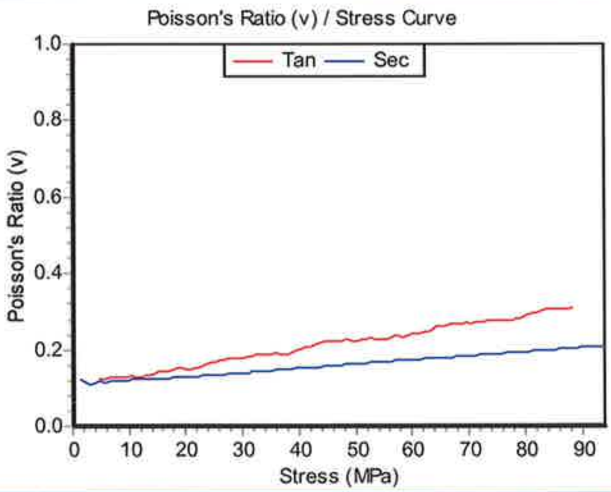
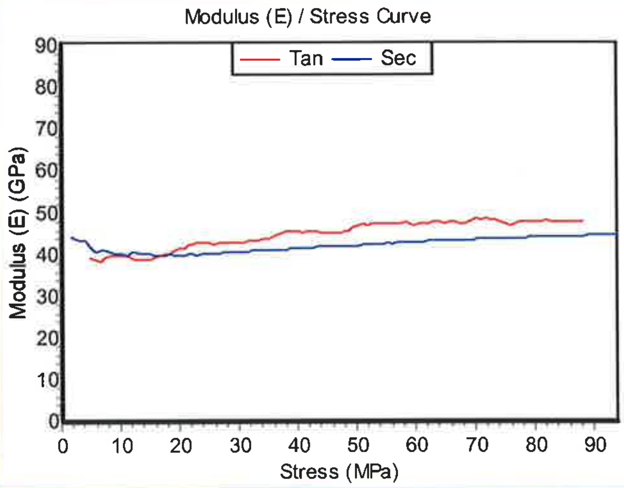
2012/07/05 11:17:47 AM

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES







Failure Load: 165.6 kN Peak Strength: 93.79 MPa Axial Strain at Failure: 2096 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	9.38	39.7	40.2	0.130	0.121
20	18.8	41	40	0.151	0.129
30	28.1	43.2	40.7	0.177	0.138
40	37.5	45.8	41.4	0.189	0.150
50	46.9	45.3	42.2	0.225	0.160
60	56.3	47.6	42.8	0.238	0.171
70	65.7	48	43.4	0.263	0.180
80	75	47.3	44	0.277	0.190
90	84.4	48.2	44.4	0.306	0.200

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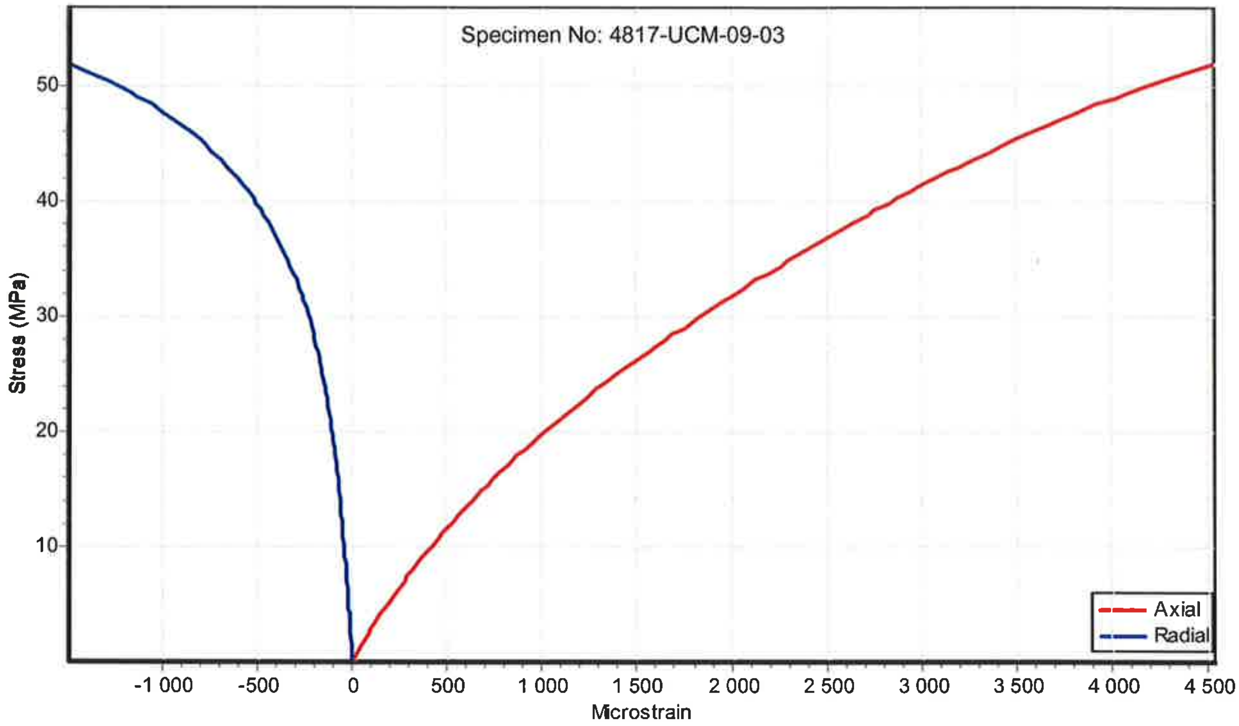
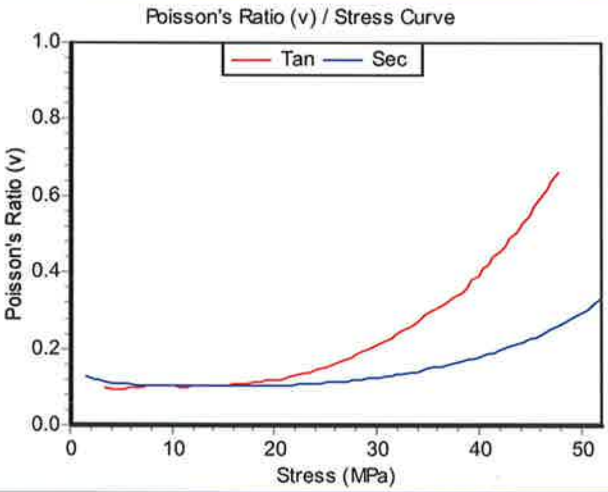
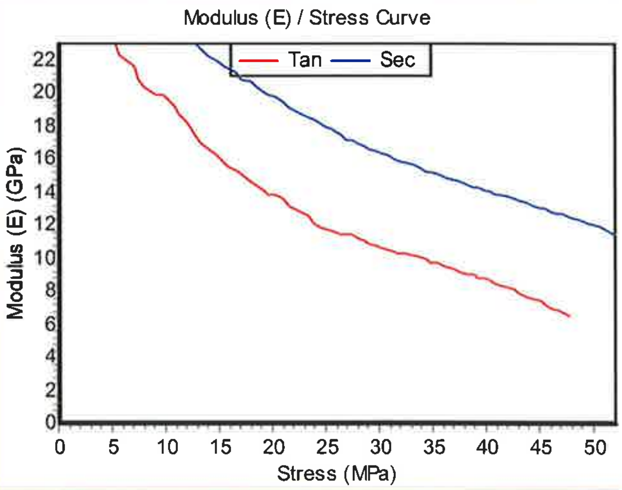
WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES







Failure Load: 161.3 kN | Peak Strength: 51.94 MPa | Axial Strain at Failure: 4529 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	5.19	23	26.5	0.095	0.108
20	10.4	19.2	23.5	0.101	0.103
30	15.6	15.6	21.5	0.107	0.103
40	20.8	13.6	19.4	0.125	0.106
50	26	11.6	17.6	0.162	0.114
60	31.2	10.4	16	0.229	0.131
70	36.4	9.38	14.8	0.323	0.159
80	41.6	8.23	13.7	0.454	0.196
90	46.7	6.7	12.7	0.639	0.251

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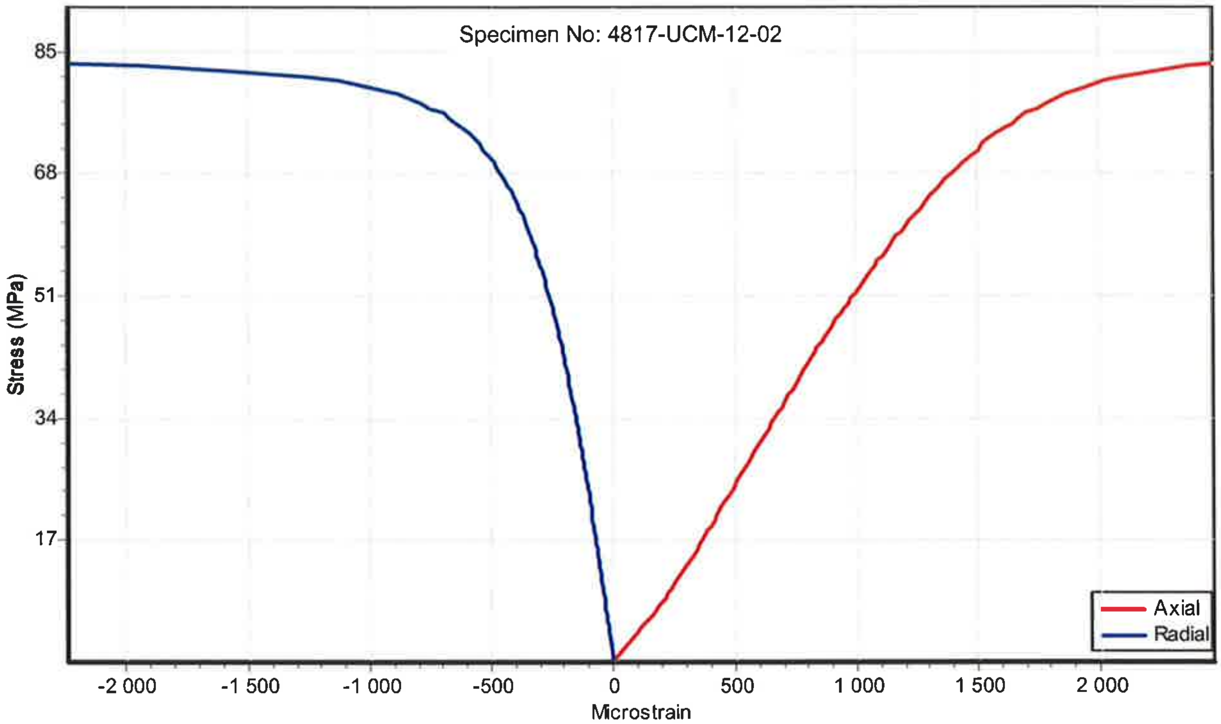
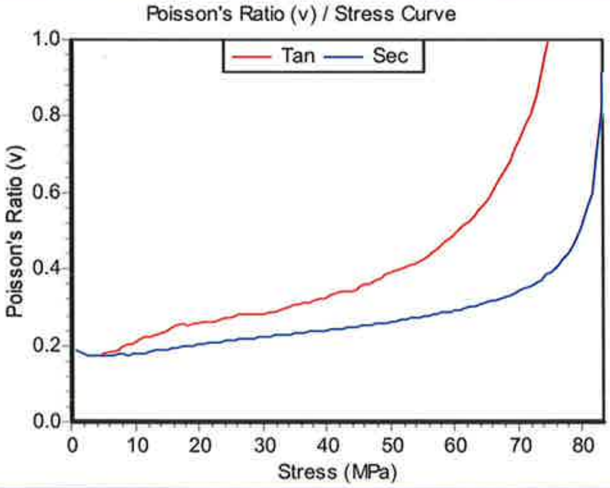
2012/07/05 11:19:33 AM

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES

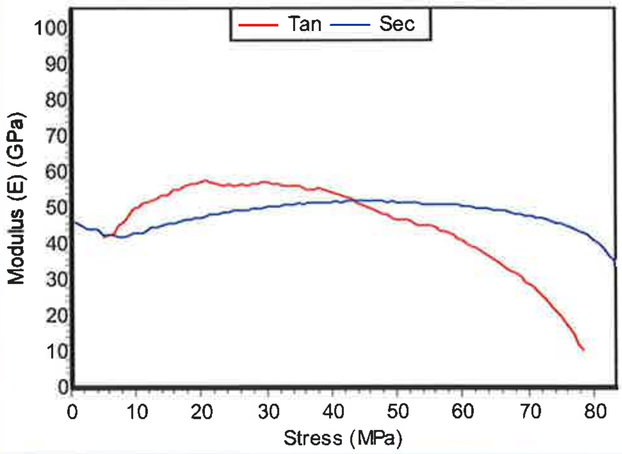
Stress / Strain Curve



Poisson's Ratio (v) / Stress Curve



Modulus (E) / Stress Curve



Failure Load: 147.5 kN | Peak Strength: 83.24 MPa | Axial Strain at Failure: 2455 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	8.32	48.5	42.6	0.201	0.174
20	16.6	56.3	46.8	0.255	0.197
30	25	56.4	49.7	0.279	0.214
40	33.3	56.5	51.3	0.303	0.230
50	41.6	53.2	52.1	0.340	0.245
60	49.9	47	51.8	0.392	0.262
70	58.3	42.6	51	0.474	0.287
80	66.6	32.9	49.1	0.633	0.323
90	74.9	20	45.7	1.019	0.391

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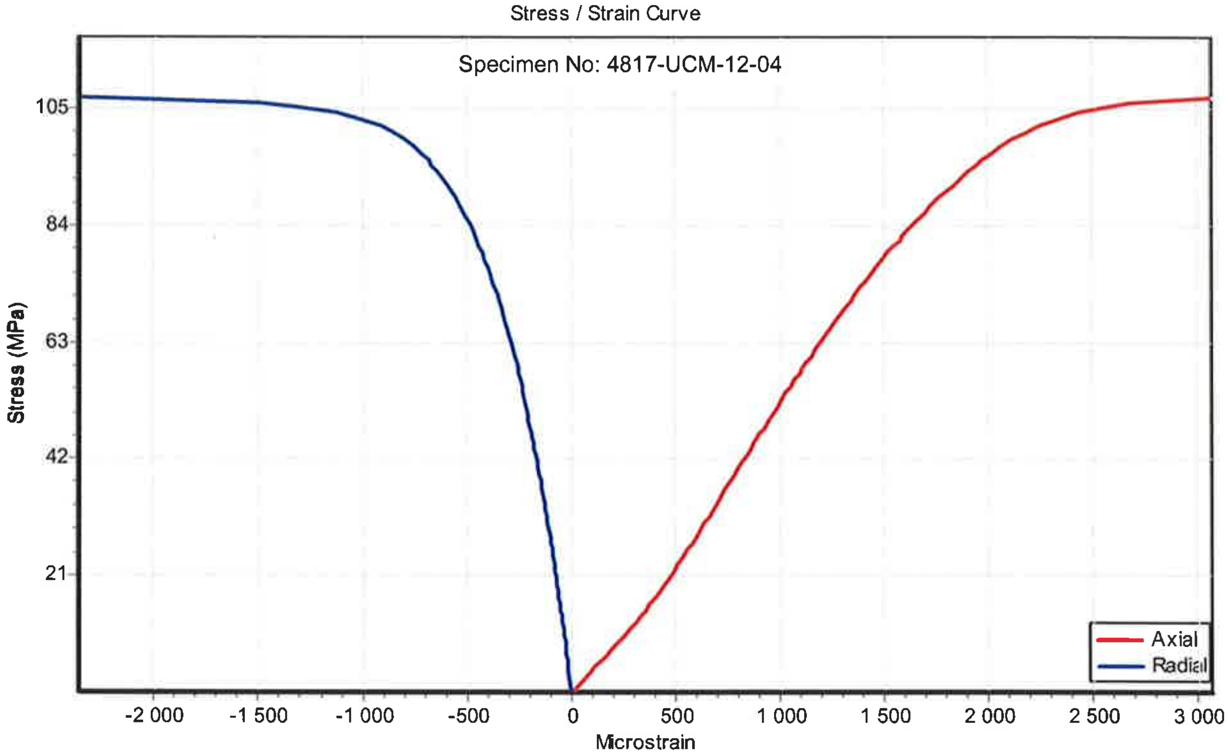
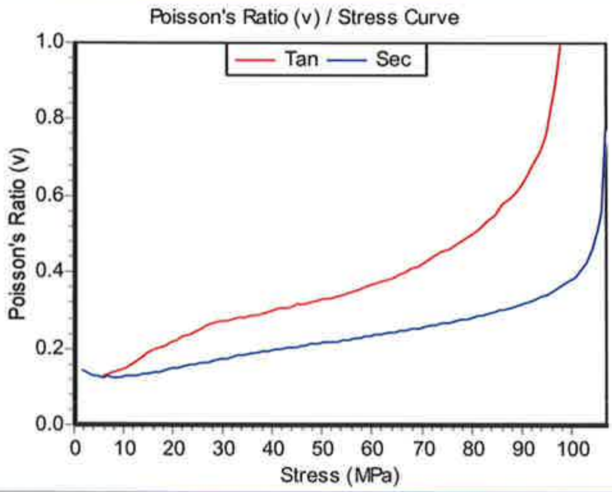
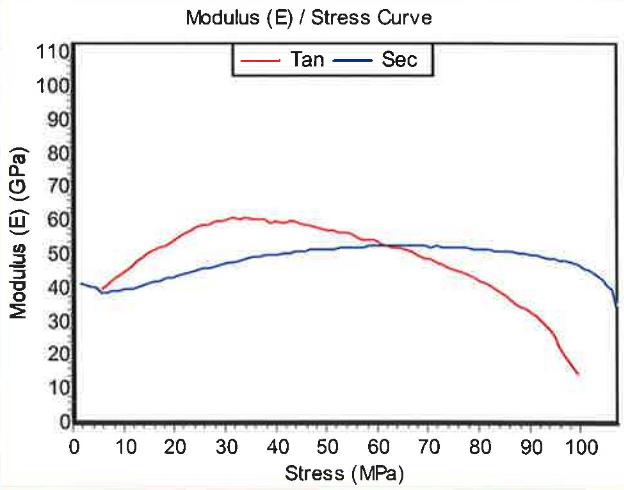
UNIAXIAL COMPRESSION TEST

2012/07/05 11:19:56 AM

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES







Failure Load: 189.3 kN   Peak Strength: 107 MPa   Axial Strain at Failure: 3062 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	10.7	46.5	40.1	0.158	0.128
20	21.4	56.4	44.3	0.232	0.154
30	32.1	60.9	48.3	0.281	0.181
40	42.8	60	50.9	0.313	0.203
50	53.5	56.7	52.3	0.341	0.223
60	64.2	52.5	52.6	0.388	0.243
70	74.9	45.7	52.3	0.459	0.269
80	85.6	37.4	50.9	0.563	0.301
90	96.3	20.3	48.1	0.896	0.354

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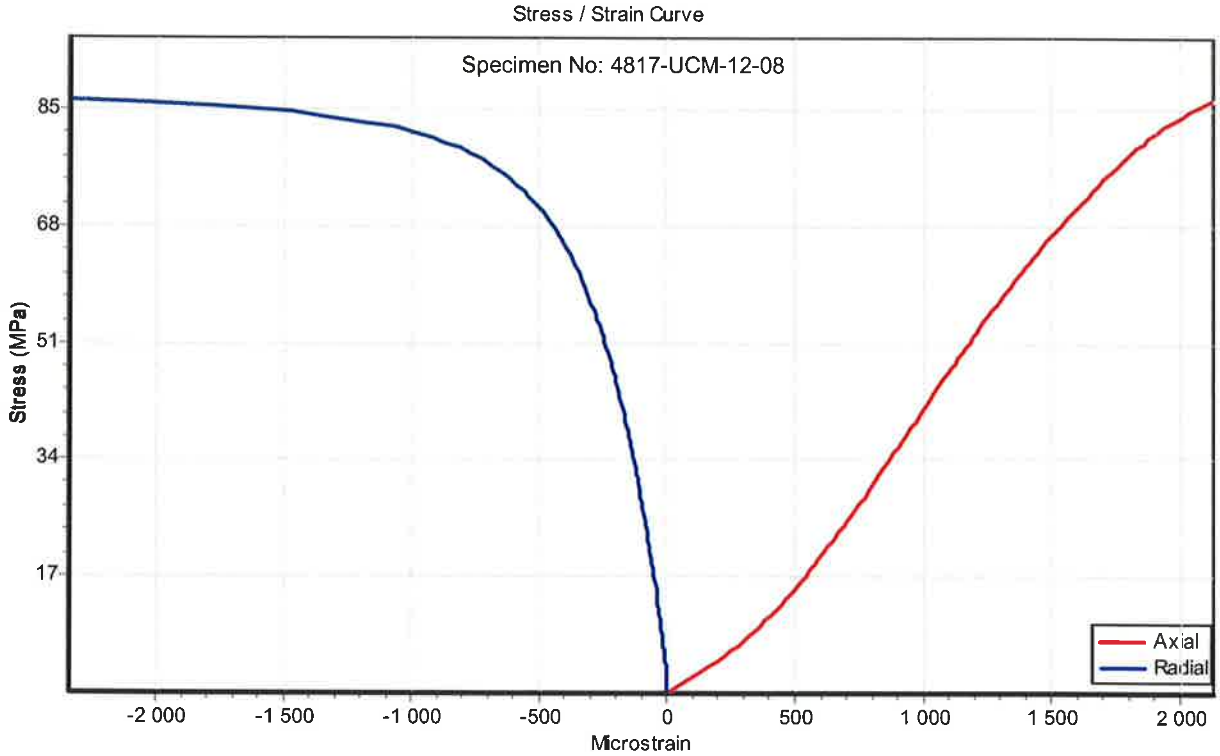
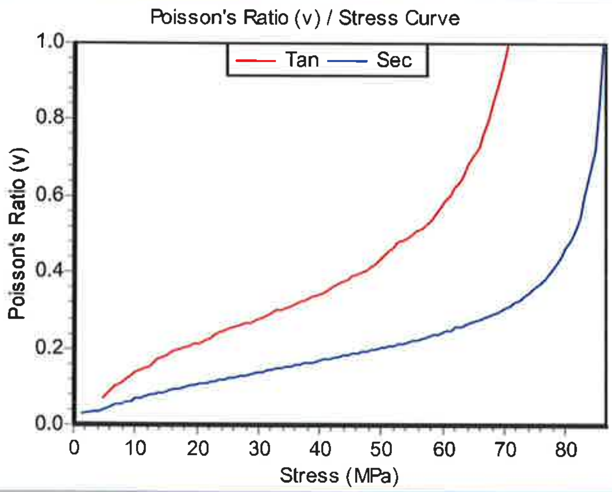
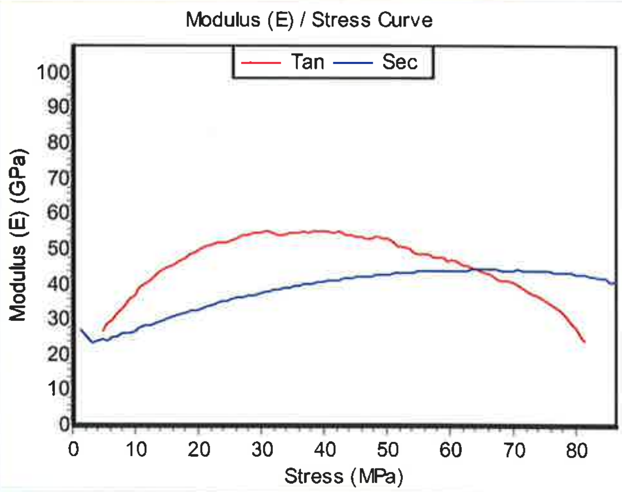
UNIAXIAL COMPRESSION TEST

2012/07/05 11:20:24 AM

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES







Failure Load: 153.8 kN   Peak Strength: 86.49 MPa   Axial Strain at Failure: 2127 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	8.65	35.8	26.5	0.129	0.061
20	17.3	47.9	32.3	0.204	0.099
30	25.9	53	36.3	0.259	0.126
40	34.6	54.9	39.7	0.311	0.154
50	43.2	54.1	42	0.375	0.181
60	51.9	51.2	43.5	0.467	0.209
70	60.5	46.8	44.3	0.606	0.250
80	69.2	41.1	44.3	0.893	0.298
90	77.8	31.2	43.5	2.219	0.425

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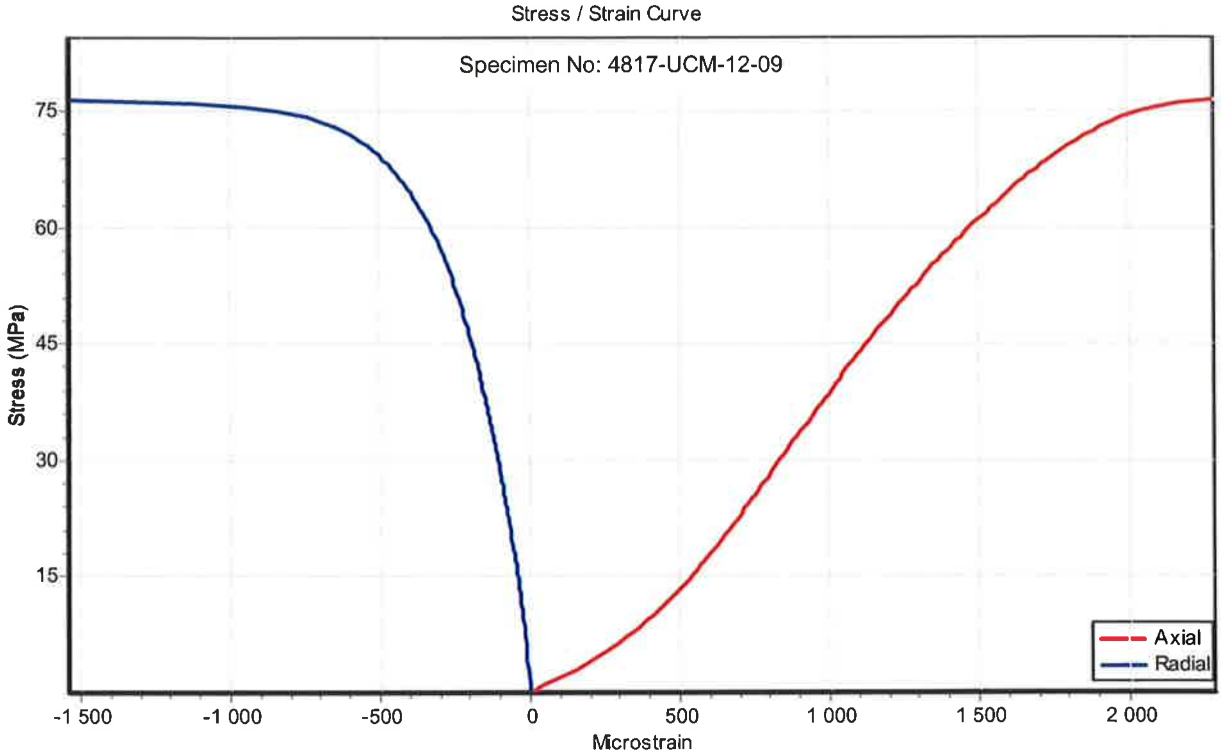
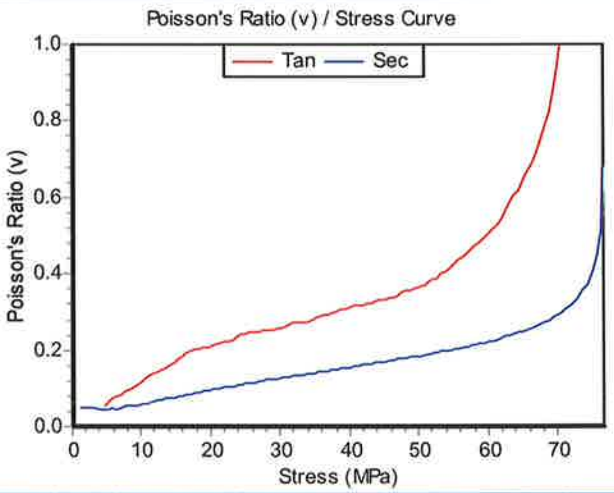
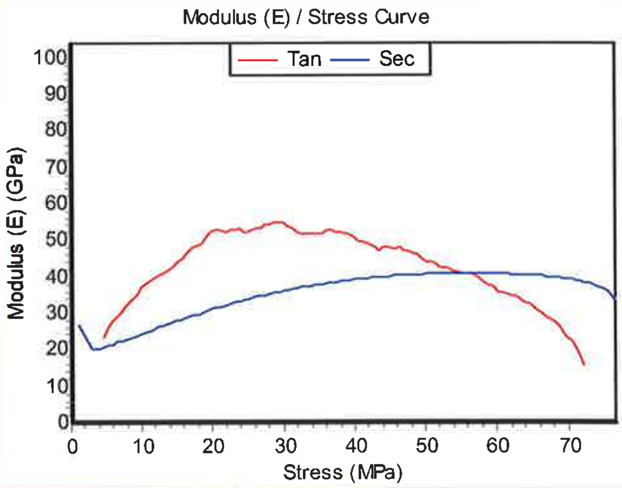
UNIAXIAL COMPRESSION TEST

2012/07/05 11:20:42 AM

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES







Failure Load: 136.1 kN   Peak Strength: 76.54 MPa   Axial Strain at Failure: 2275 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	7.65	32.3	22.6	0.092	0.052
20	15.3	45.2	28.5	0.183	0.080
30	23	52.7	33	0.229	0.106
40	30.6	53.9	36.3	0.264	0.129
50	38.3	51.8	38.7	0.305	0.152
60	45.9	47.9	40.2	0.337	0.174
70	53.6	41.5	40.9	0.406	0.198
80	61.2	35	40.7	0.529	0.230
90	68.9	25	39.5	0.873	0.285

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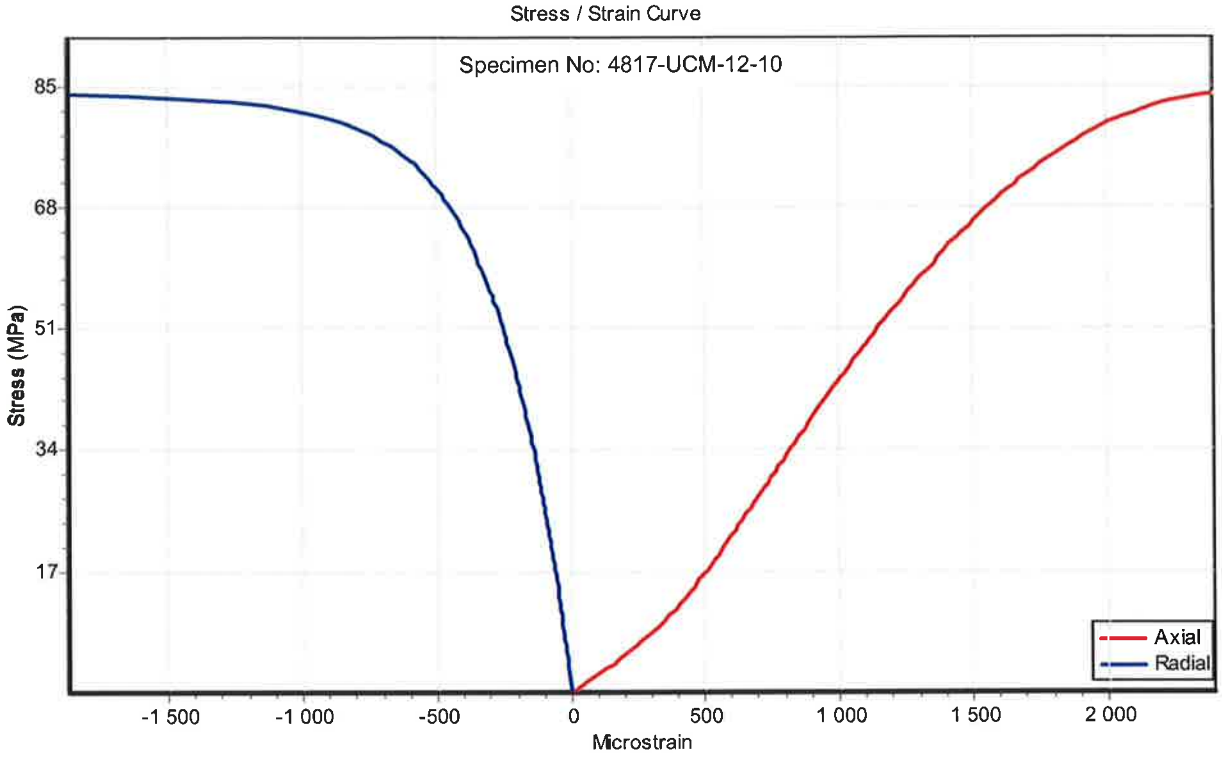
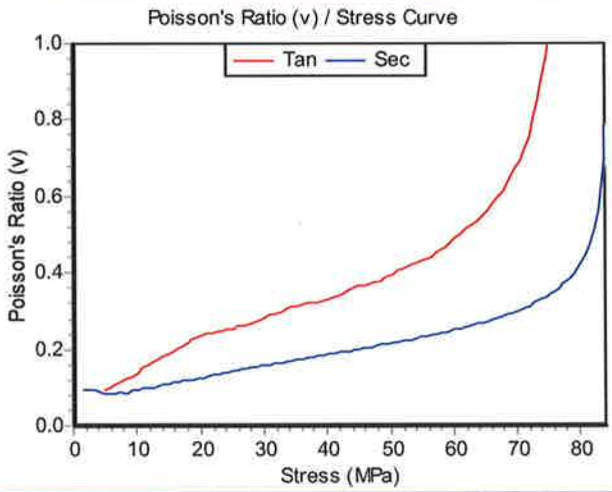
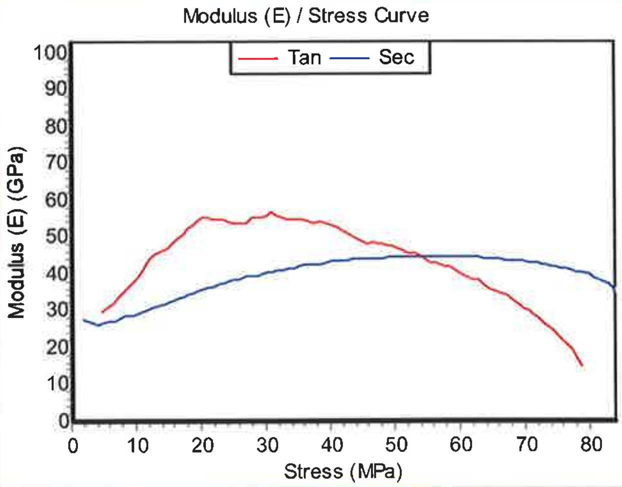
UNIAXIAL COMPRESSION TEST

2012/07/05 11:21:02 AM

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES







Failure Load: 149.2 kN | Peak Strength: 83.77 MPa | Axial Strain at Failure: 2387 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	8.38	37.4	28.8	0.131	0.093
20	16.8	50.7	33.8	0.209	0.117
30	25.1	53.8	38.5	0.260	0.145
40	33.5	55	41.5	0.305	0.167
50	41.9	51.7	43.5	0.341	0.191
60	50.3	47.4	44.4	0.396	0.217
70	58.6	41.2	44.5	0.478	0.247
80	67	33.8	43.8	0.612	0.285
90	75.4	22.3	41.7	1.055	0.346

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APPENDIX 2

GRAPHS OF BASIC FRICTION ANGLE TESTS BASED

ON DIRECT SHAER TESTS ON ROCK SAW-CUTTING SURFACE

USING ROCKLAB'S SERVO-CONTROLLED

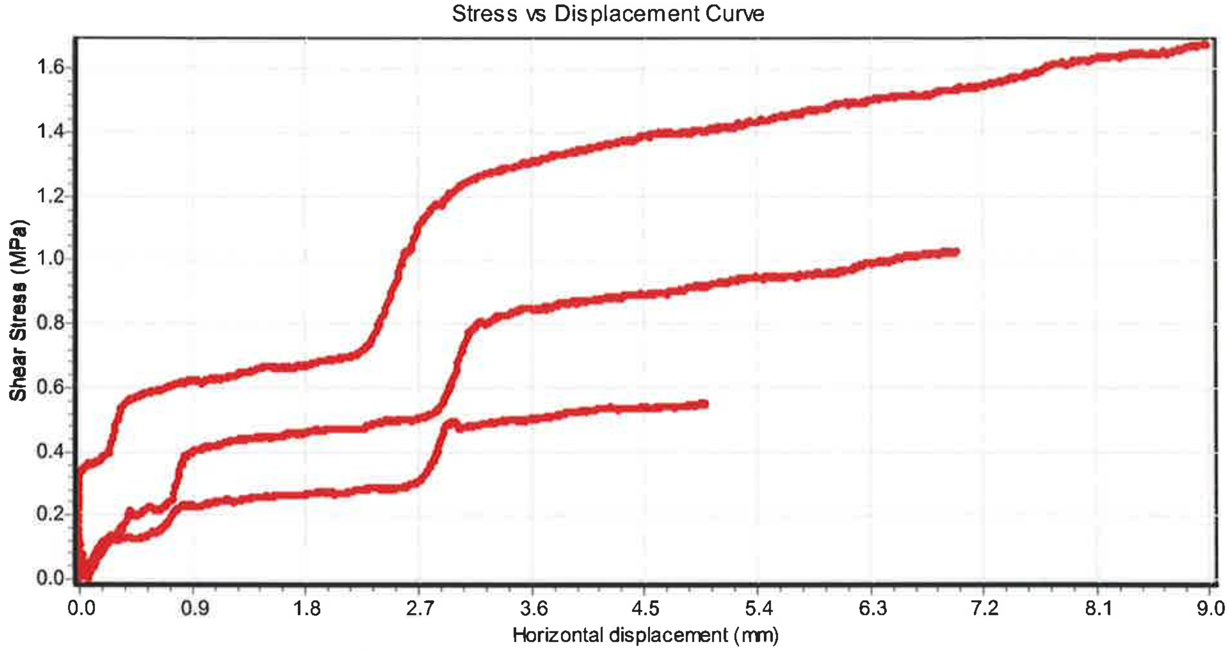
COMPUTERIZED DIRECT SHEAR TESTER

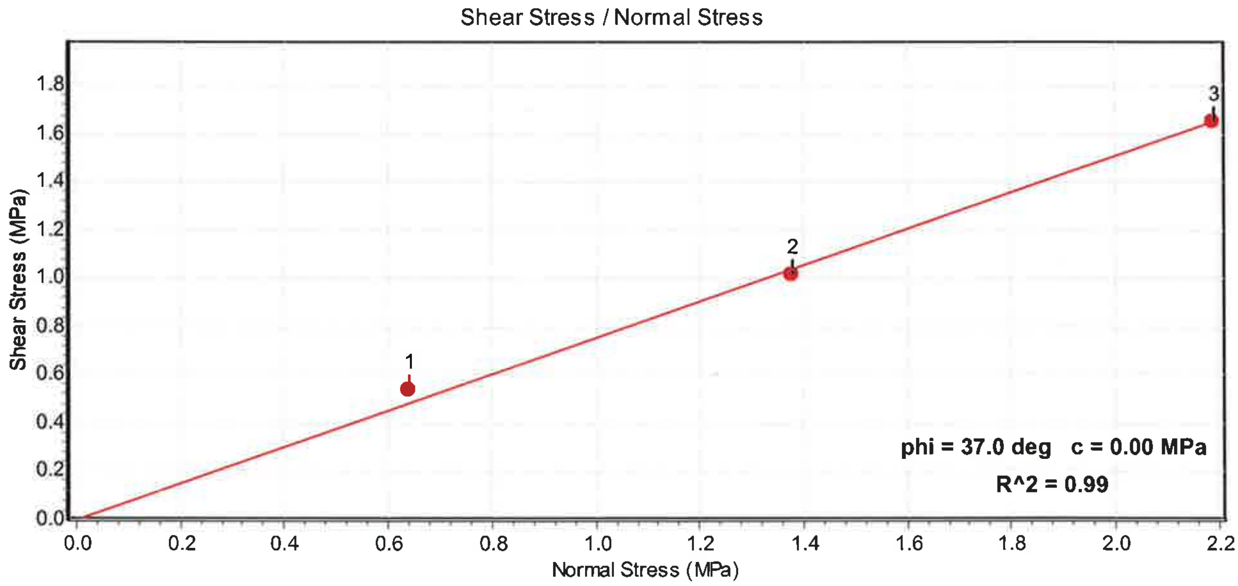
GCTS SHEAR TEST

2012/05/28 01:14:59 PM

ON AN A SAW CUT JOINT - BASIC FRICTION

4817-bfa-05-03-a, 4817-bfa-05-03-b, 4817-bfa-05-03-c





Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	4.61	1.00	0.83	0.0	1554	0.64	0.54	39.8
2	6.80	2.00	1.48	0.0	1450	1.38	1.02	36.4
3	8.52	3.00	2.25	0.0	1370	2.19	1.65	36.9

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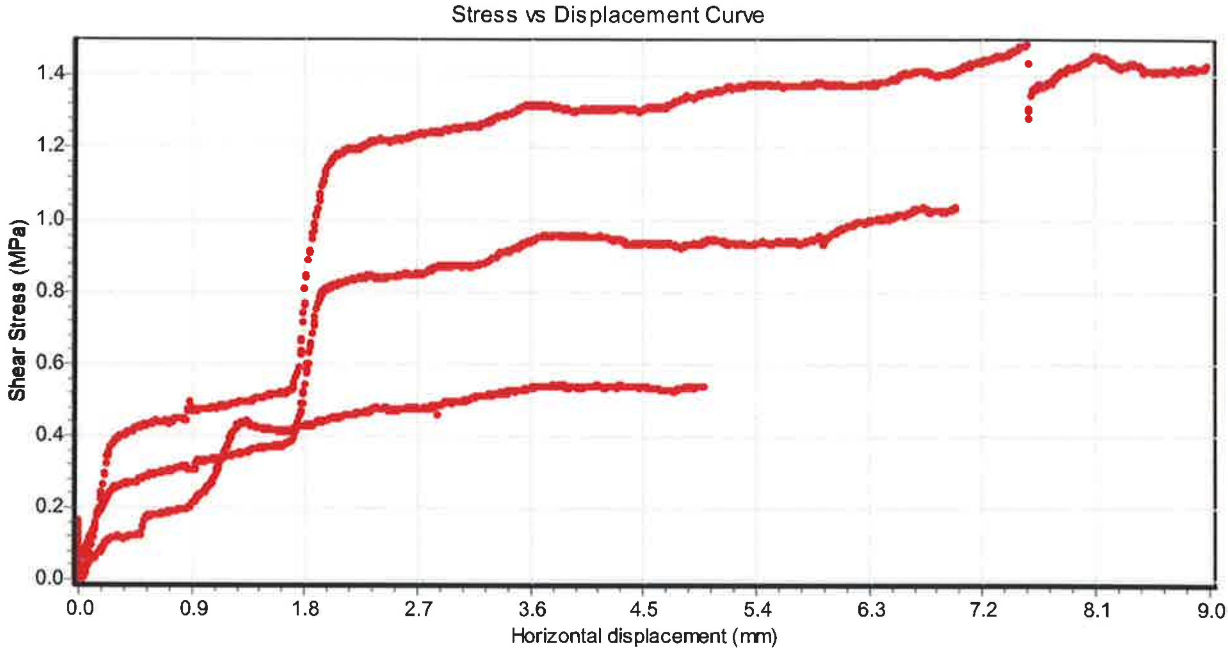
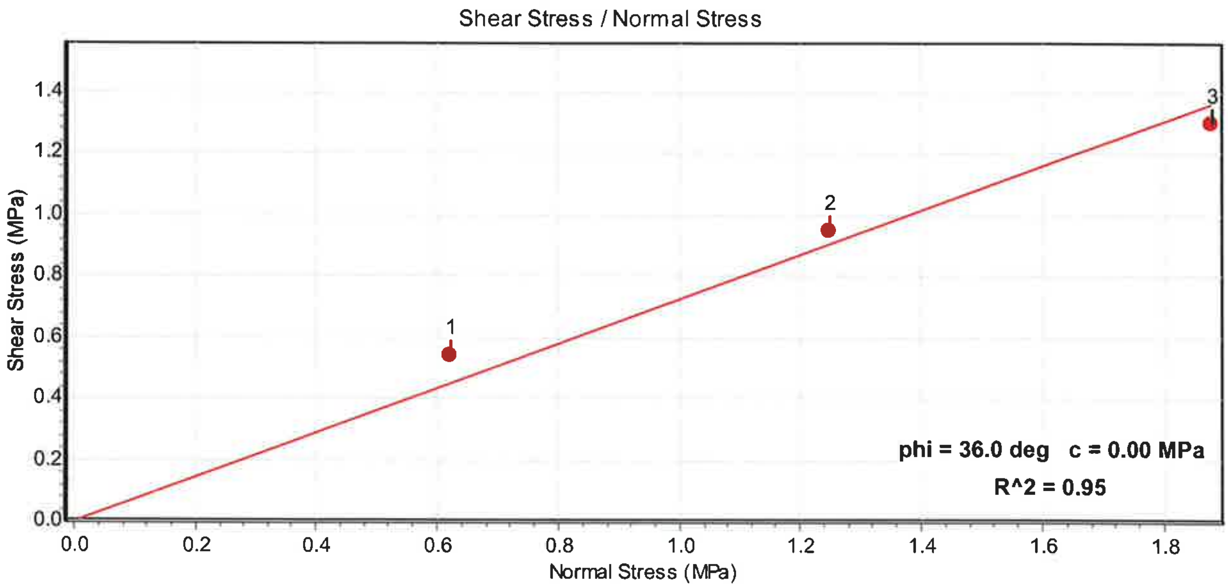
GCTS SHEAR TEST

ON AN A SAW CUT JOINT - BASIC FRICTION

2012/05/28 01:16:23 PM

4817-bfa-05-05-a, 4817-bfa-05-05-b, 4817-bfa-05-05-c





Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	3.87	1.00	0.86	0.0	1602	0.63	0.54	40.8
2	3.91	2.00	1.52	0.0	1600	1.25	0.95	37.3
3	4.07	3.00	2.08	0.0	1592	1.89	1.30	34.7

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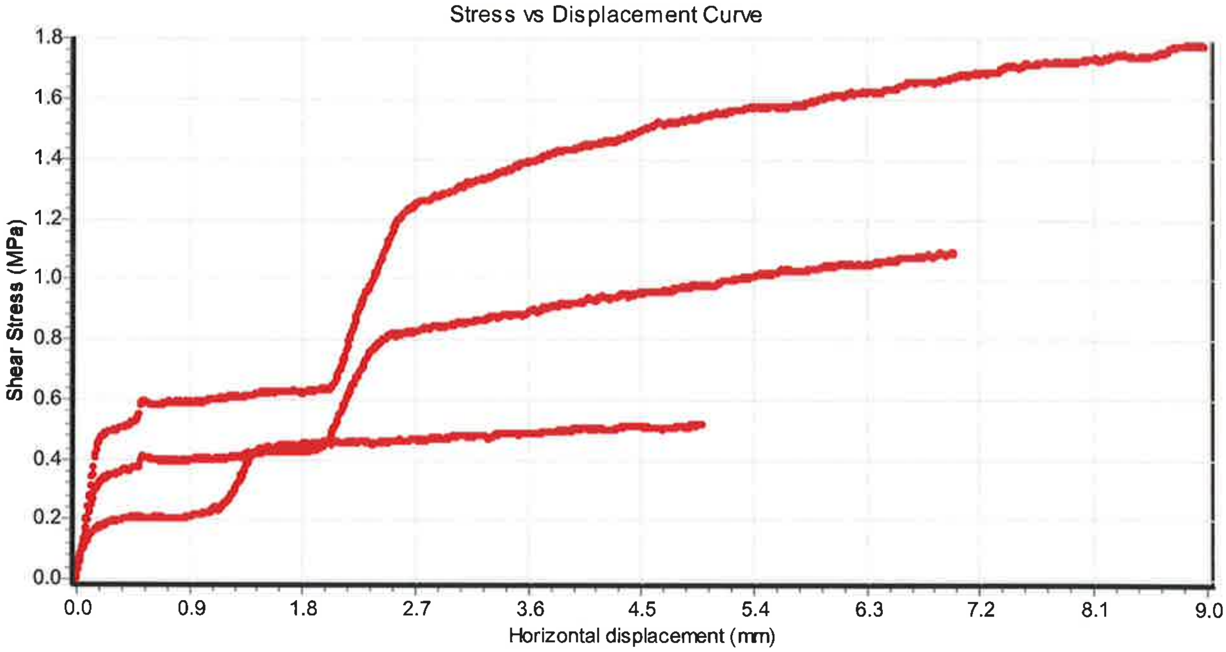
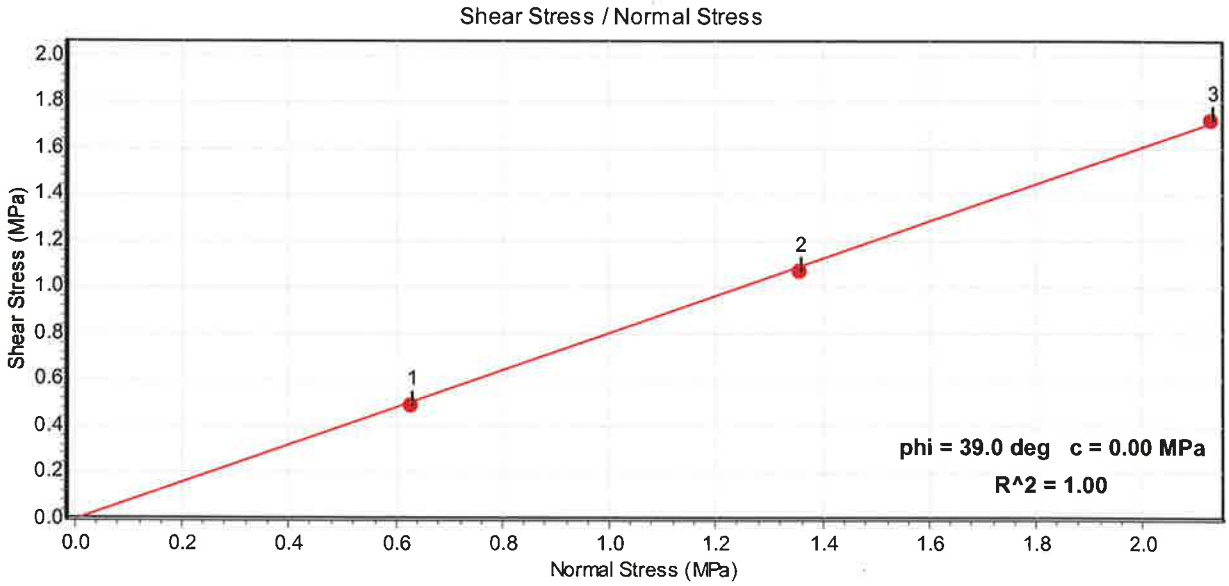
GCTS SHEAR TEST

2012/05/28 01:17:05 PM

ON AN A SAW CUT JOINT - BASIC FRICTION

4817-bfa-06-02-a, 4817-bfa-06-02-b, 4817-bfa-06-02-c





Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	3.79	1.00	0.79	0.0	1598	0.63	0.49	38.2
2	6.59	2.00	1.57	0.0	1466	1.36	1.07	38.1
3	7.74	3.00	2.43	0.0	1411	2.13	1.72	39.0

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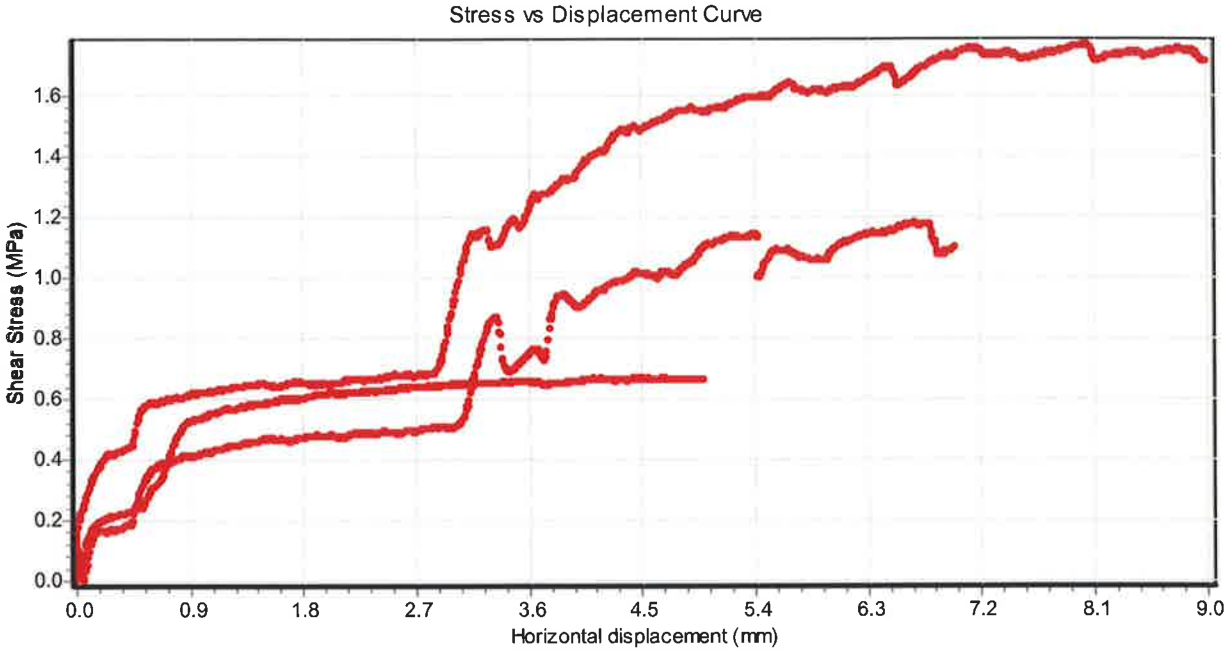
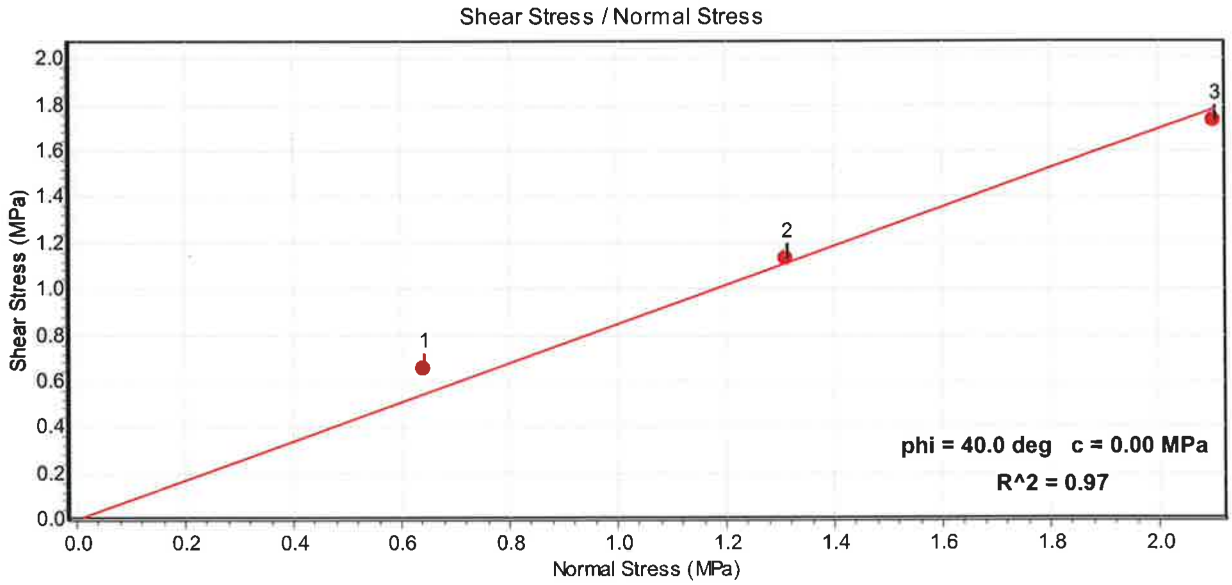
GCTS SHEAR TEST

2012/05/28 01:18:28 PM

ON AN A SAW CUT JOINT - BASIC FRICTION

4817-bfa-06-08-a, 4817-bfa-06-08-b, 4817-bfa-06-08-c





Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	4.24	1.00	1.04	0.0	1572	0.64	0.67	46.3
2	5.27	2.00	1.73	0.0	1524	1.31	1.13	40.8
3	7.30	3.00	2.47	0.0	1428	2.10	1.73	39.5

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GCTS SHEAR TEST

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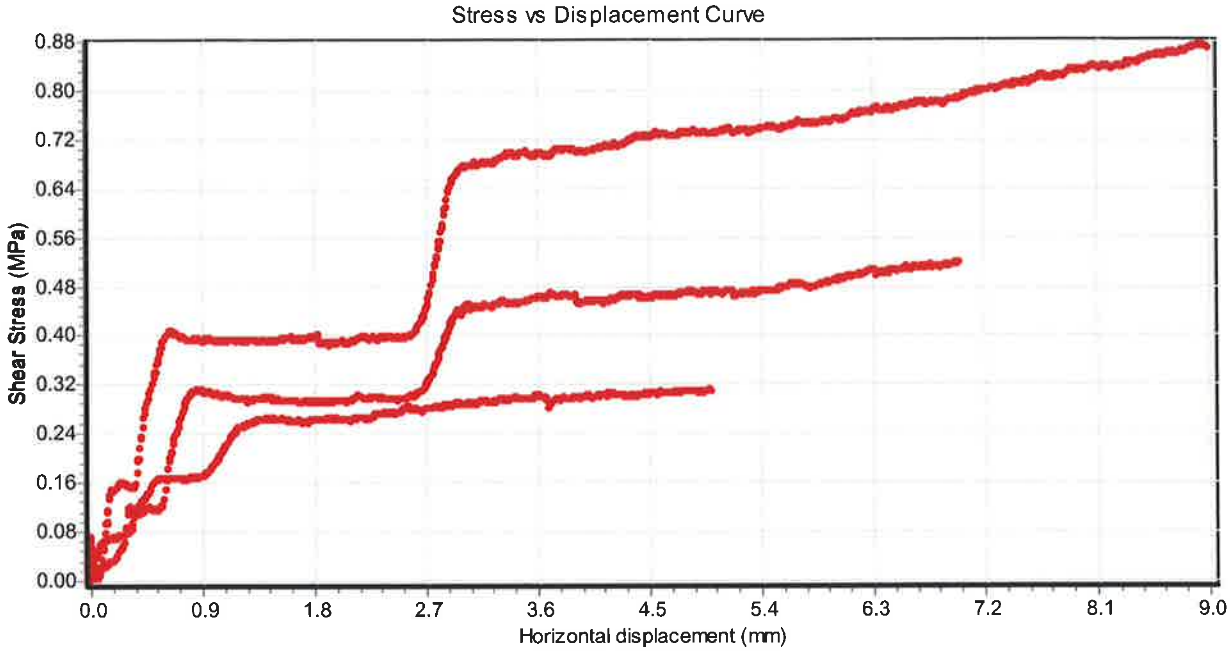
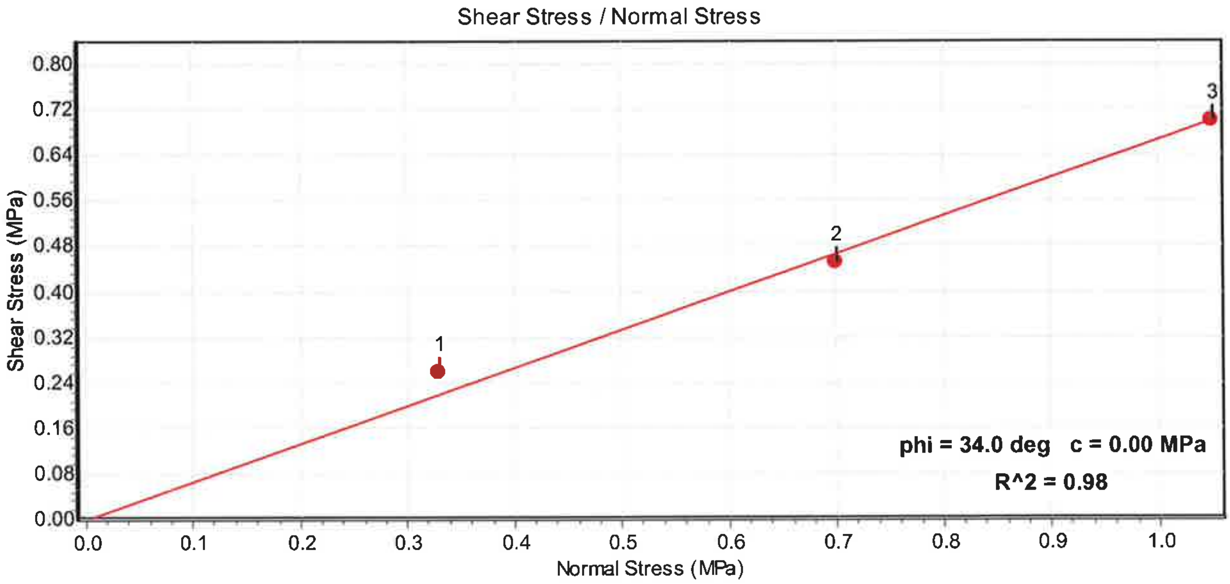
ON AN A SAW CUT JOINT - BASIC FRICTION

4817-bfa-07-08-a, 4817-bfa-07-08-b, 4817-bfa-07-08-c





Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	1.81	1.00	0.79	0.0	3007	0.33	0.26	38.4
2	4.12	2.00	1.30	0.0	2862	0.70	0.45	33.0
3	4.02	3.00	2.01	0.0	2868	1.05	0.70	33.8

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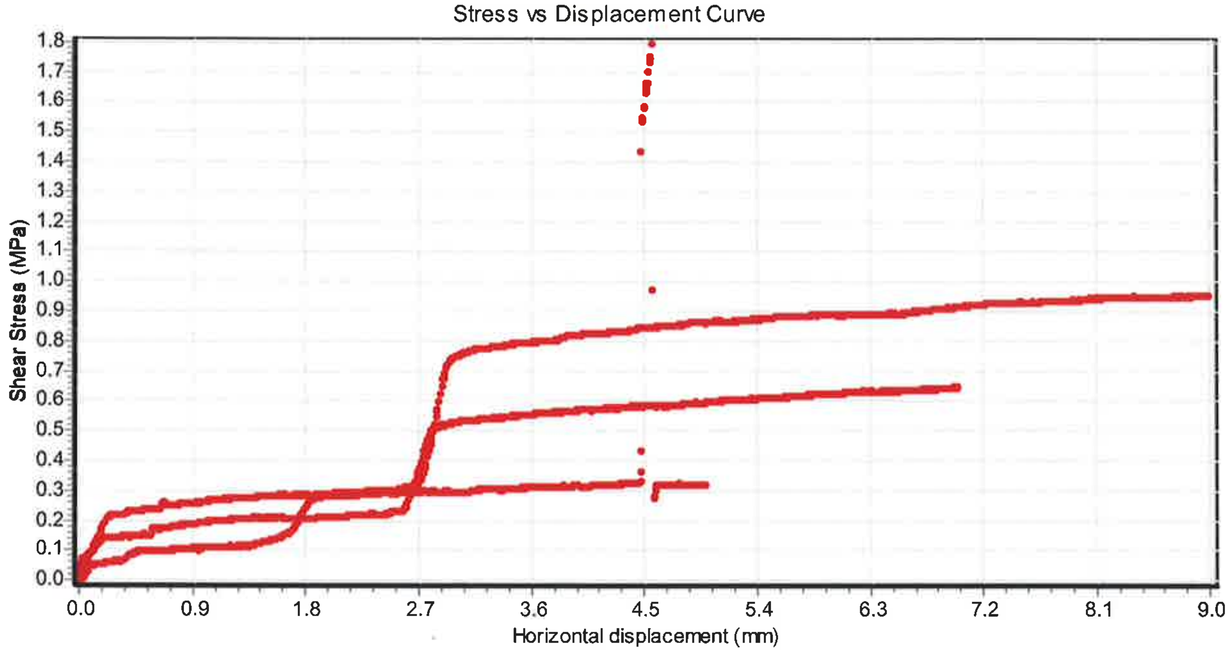
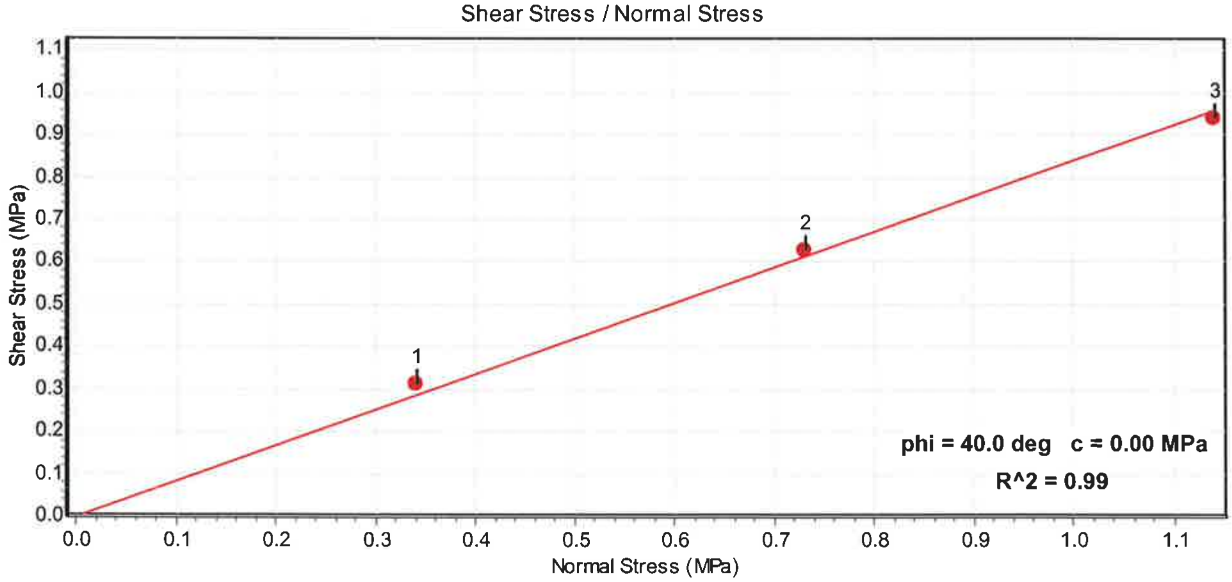
GCTS SHEAR TEST

2012/05/28 01:21:00 PM

ON AN A SAW CUT JOINT - BASIC FRICTION

4817-bfa-07-10-a, 4817-bfa-07-10-b, 4817-bfa-07-10-c





Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	3.52	1.00	0.89	0.0	2903	0.34	0.31	41.6
2	6.28	2.00	1.72	0.0	2730	0.73	0.63	40.7
3	8.00	3.00	2.46	0.0	2622	1.14	0.94	39.3

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GCTS SHEAR TEST

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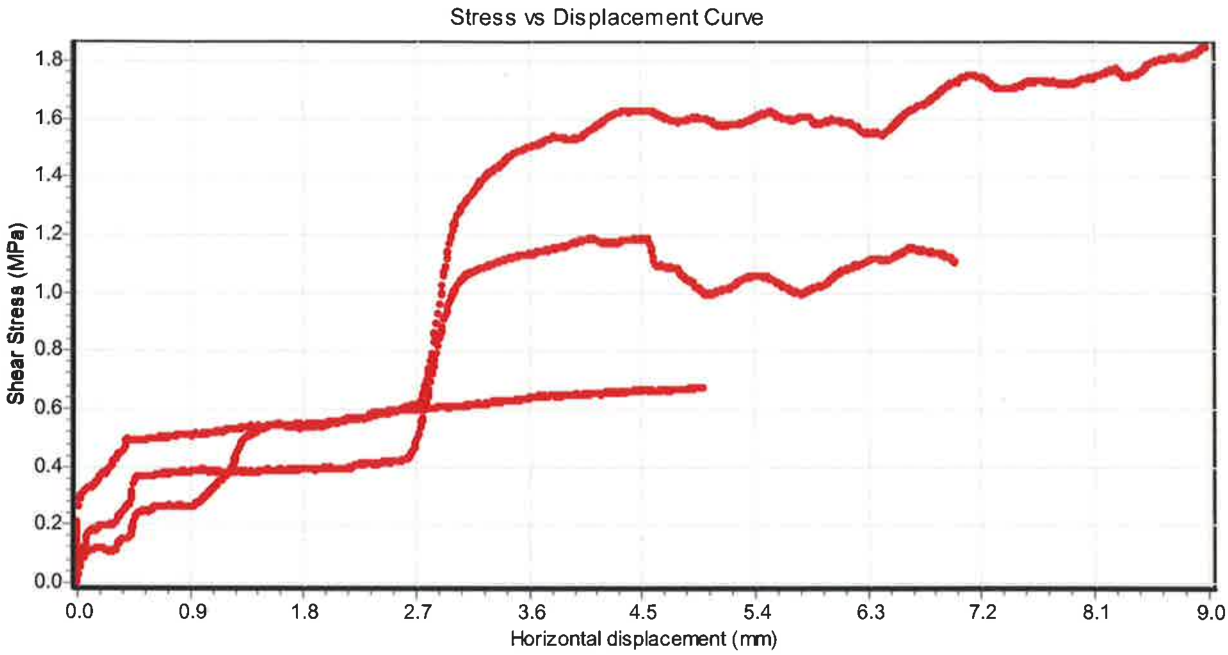
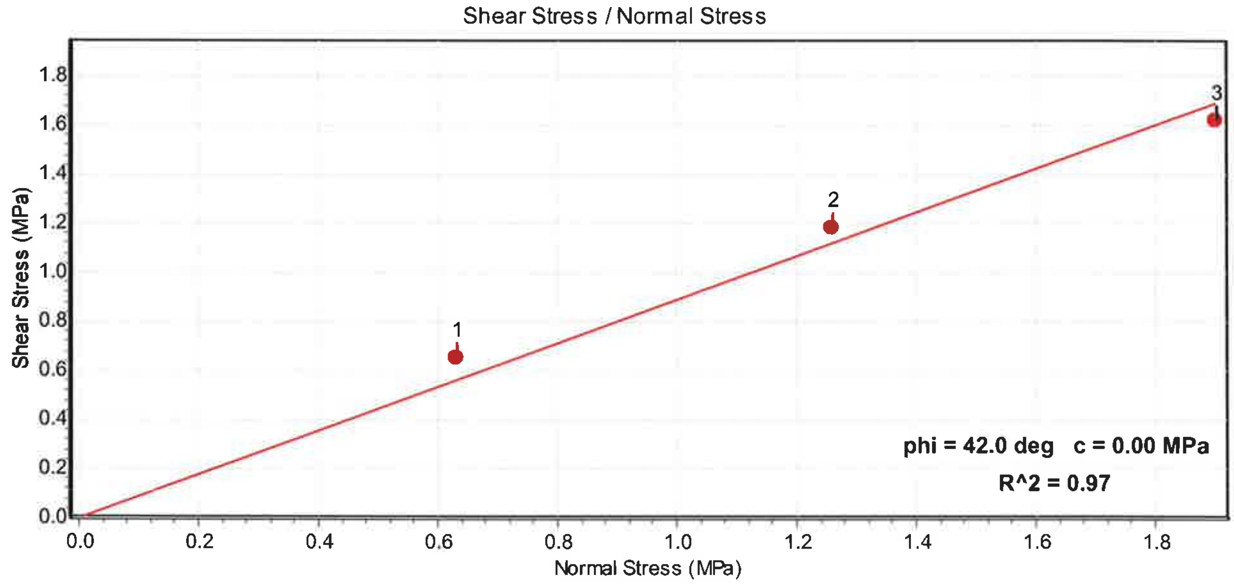
ON AN A SAW CUT JOINT - BASIC FRICTION

4817-bfa-07-15-a, 4817-bfa-07-15-b, 4817-bfa-07-15-c





Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	4.09	1.00	1.04	0.0	1596	0.63	0.65	46.1
2	4.35	2.00	1.86	0.0	1583	1.26	1.18	43.0
3	4.48	3.00	2.56	0.0	1577	1.90	1.62	40.4

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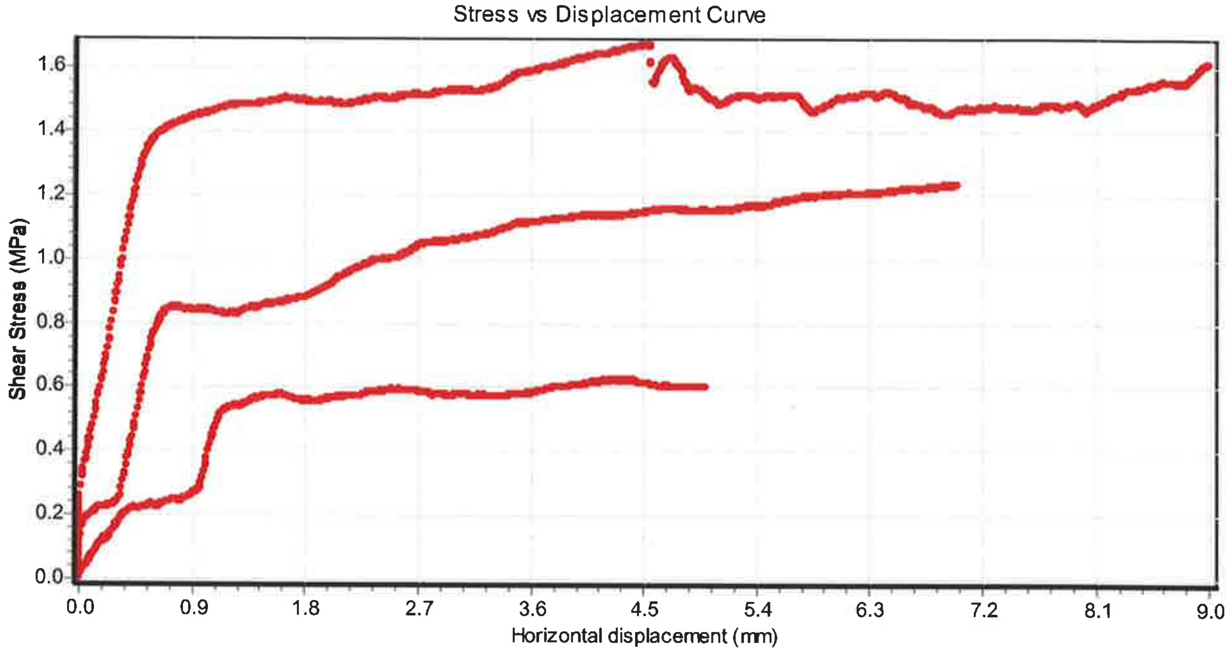
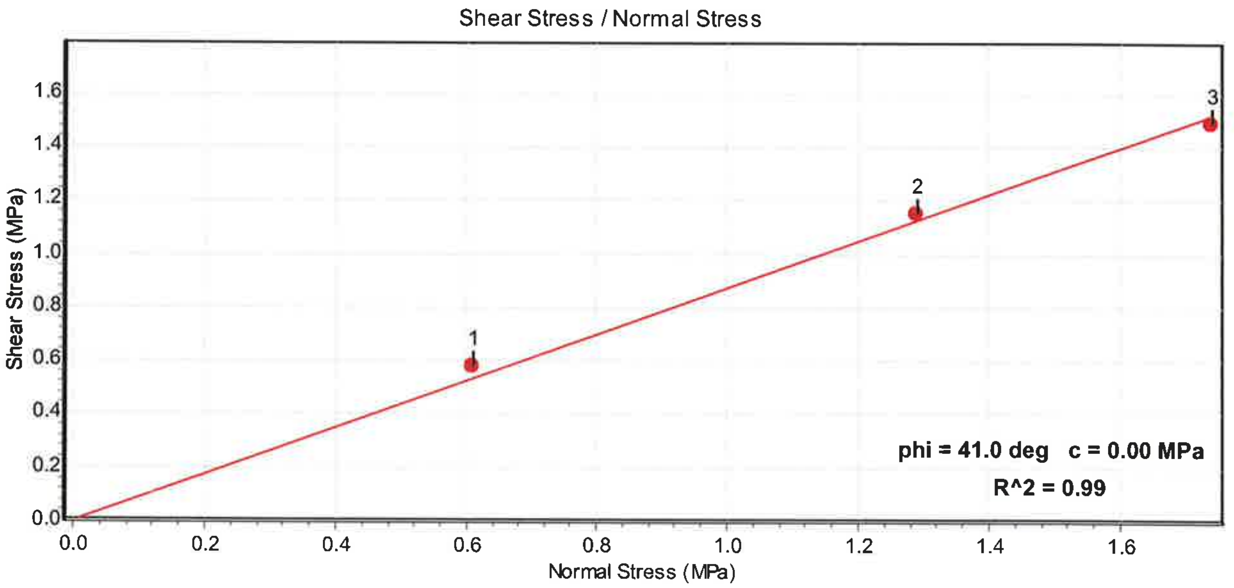
GCTS SHEAR TEST

2012/05/28 01:22:37 PM

ON AN A SAW CUT JOINT - BASIC FRICTION

4817-bfa-07-36-a, 4817-bfa-07-36-b, 4817-bfa-07-36-c





Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	3.09	1.00	0.95	0.0	1647	0.61	0.58	43.5
2	5.01	2.00	1.79	0.0	1555	1.29	1.15	41.9
3	1.49	3.00	2.56	0.0	1723	1.74	1.49	40.5

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GCTS SHEAR TEST

2012/05/28 01:23:16 PM

ON AN A SAW CUT JOINT - BASIC FRICTION

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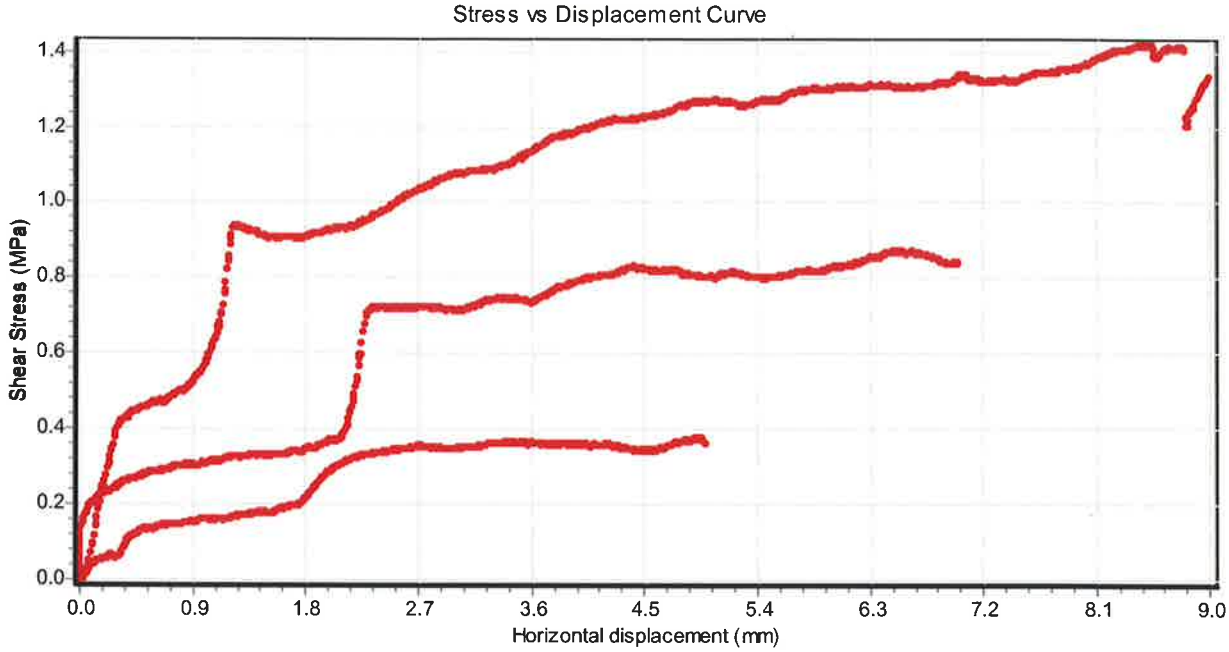

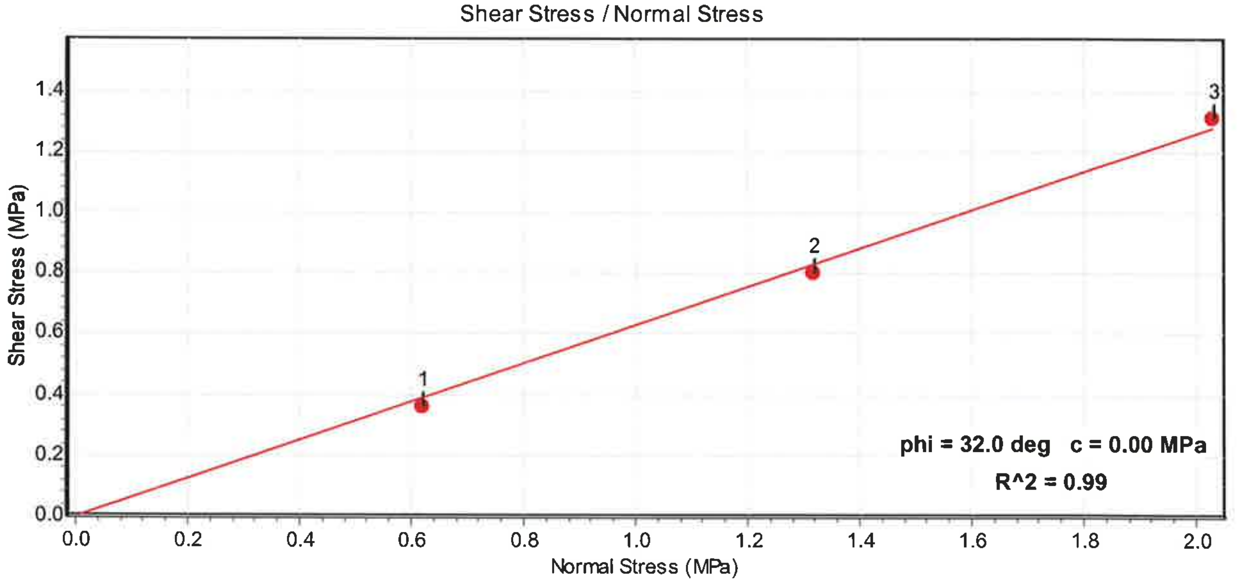

Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	3.50	1.00	0.58	0.0	1606	0.62	0.36	30.1
2	5.36	2.00	1.22	0.0	1518	1.32	0.80	31.3
3	6.15	3.00	1.94	0.0	1481	2.03	1.31	32.8

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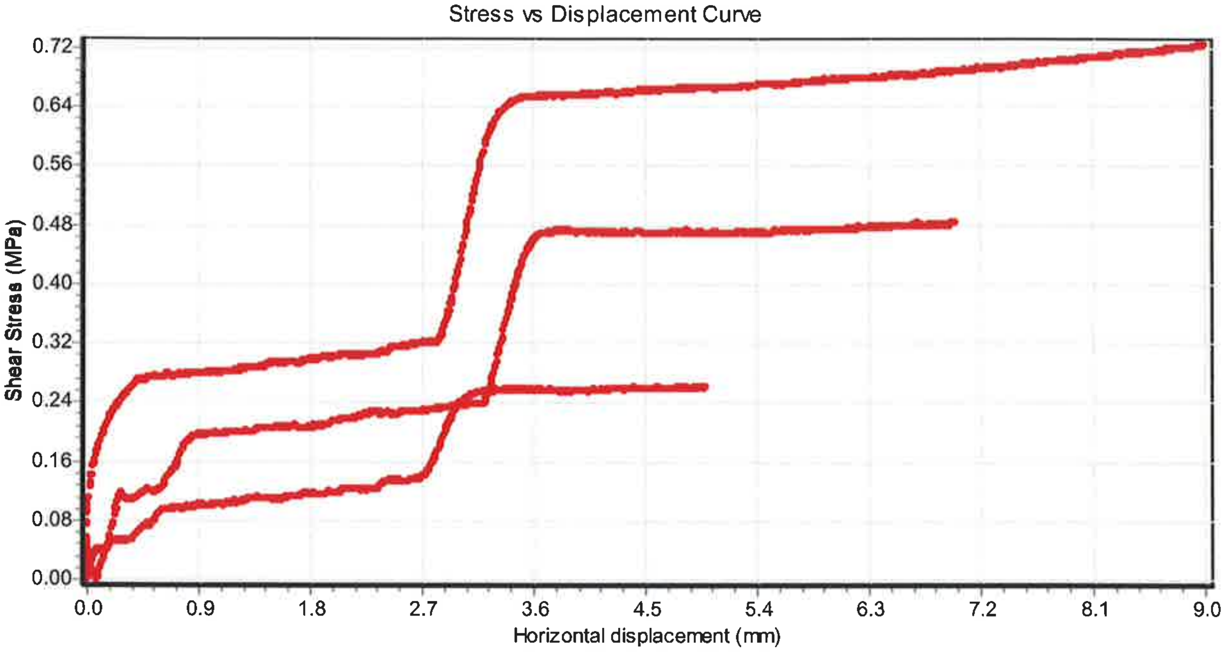
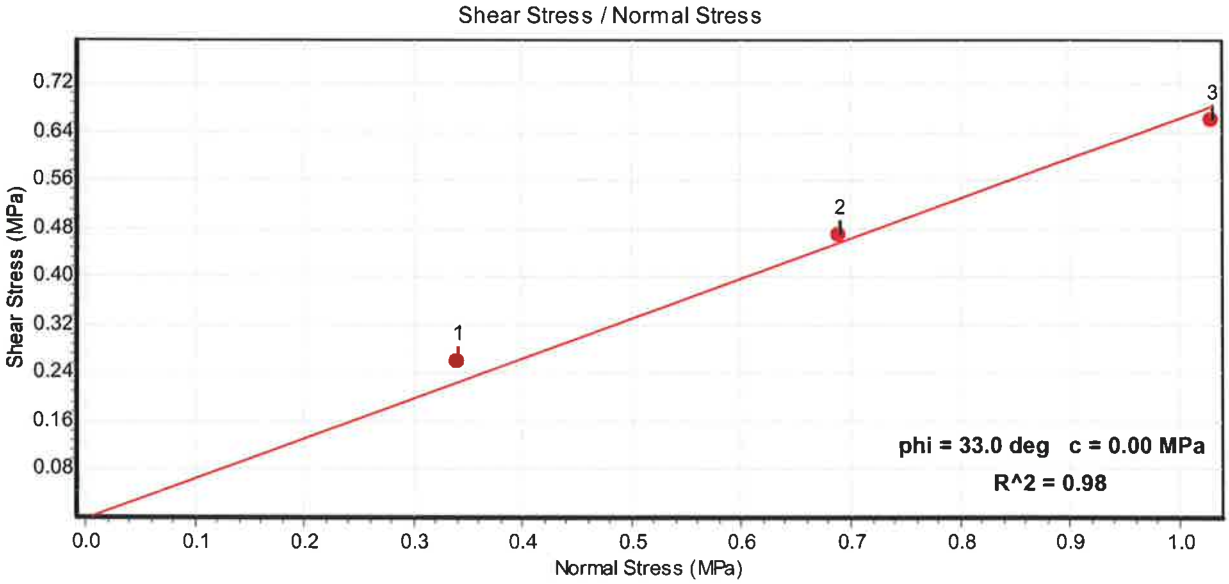
2012/05/28 01:23:52 PM

GCTS SHEAR TEST

ON AN A SAW CUT JOINT - BASIC FRICTION

4817-bfa-08-02-a, 4817-bfa-08-02-b, 4817-bfa-08-02-c





Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	4.28	1.00	0.76	0.0	2920	0.34	0.26	37.1
2	4.58	2.00	1.36	0.0	2901	0.69	0.47	34.2
3	4.55	3.00	1.93	0.0	2906	1.03	0.66	32.7

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GCTS SHEAR TEST

2012/05/28 01:24:18 PM

ON AN A SAW CUT JOINT - BASIC FRICTION

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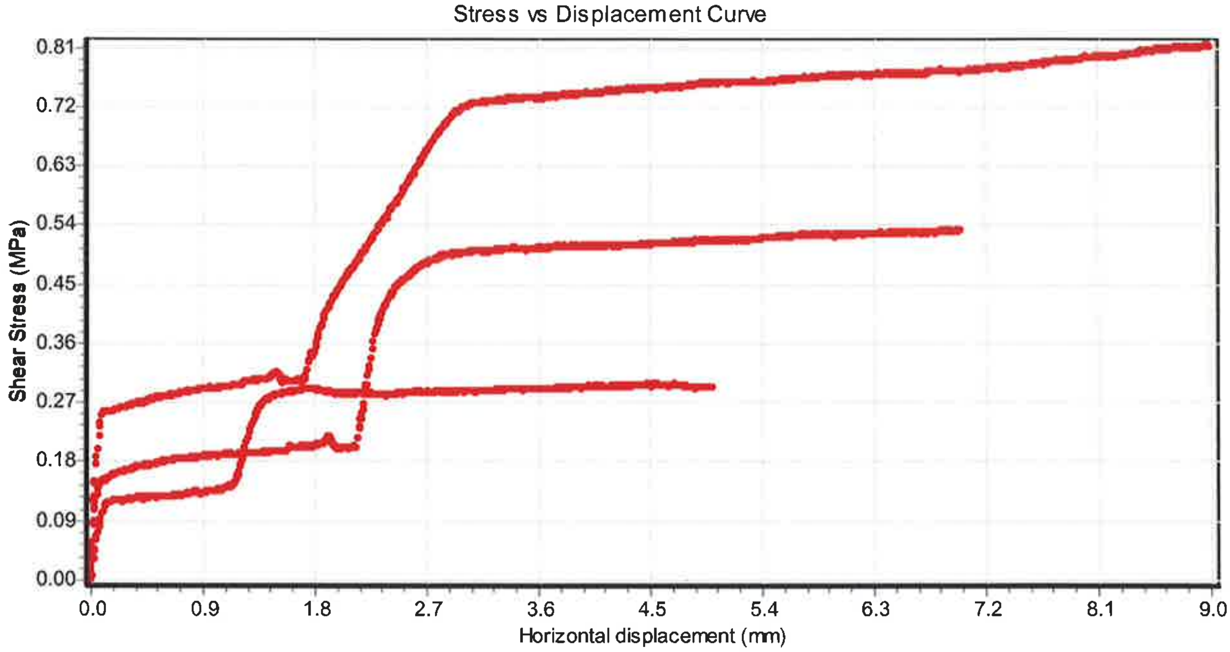

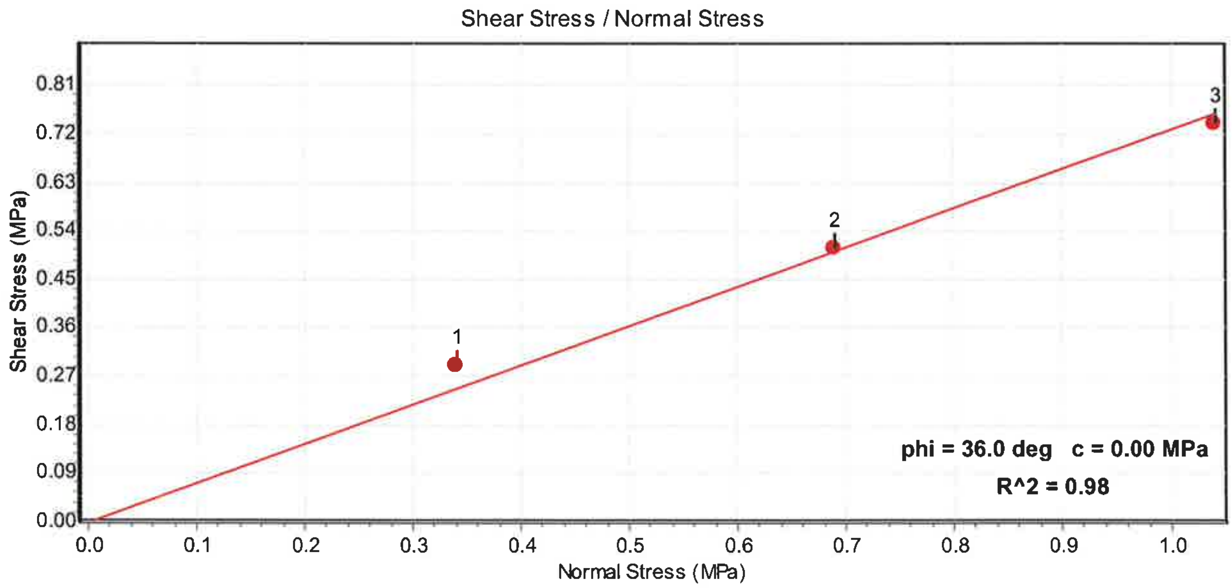

Selected Points

No	Horiz. Displ. (mm)	Vertical Load (kN)	Horizontal Load (kN)	Dilatancy Angle (deg)	Contact Area (mm^2)	Normal Stress (MPa)	Shear Stress (MPa)	App. Friction Angle (deg)
1	3.30	1.00	0.85	0.0	2930	0.34	0.29	40.3
2	3.88	2.00	1.47	0.0	2894	0.69	0.51	36.2
3	4.20	3.00	2.14	0.0	2874	1.04	0.74	35.5

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APPENDIX 3

CLASSIFICATION OF ROCK SPECIMEN FAILURE MODE INFLUENCED / NOT INFLUENCED BY DISCONTINUITIES DURING COMPRESSION TESTING

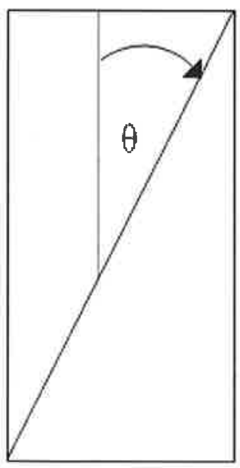
FAILURE NOT INFLUENCED BY DISCONTINUITIES (INTACT)

TYPE CODE	DESCRIPTION OF SUB CODES	
	A	B
X	SLIDING SHEAR FAILURE	COMPLETE CONE DEVELOPMENT
Y	SPLITTING	BREAKING INTO A LOT OF PIECES

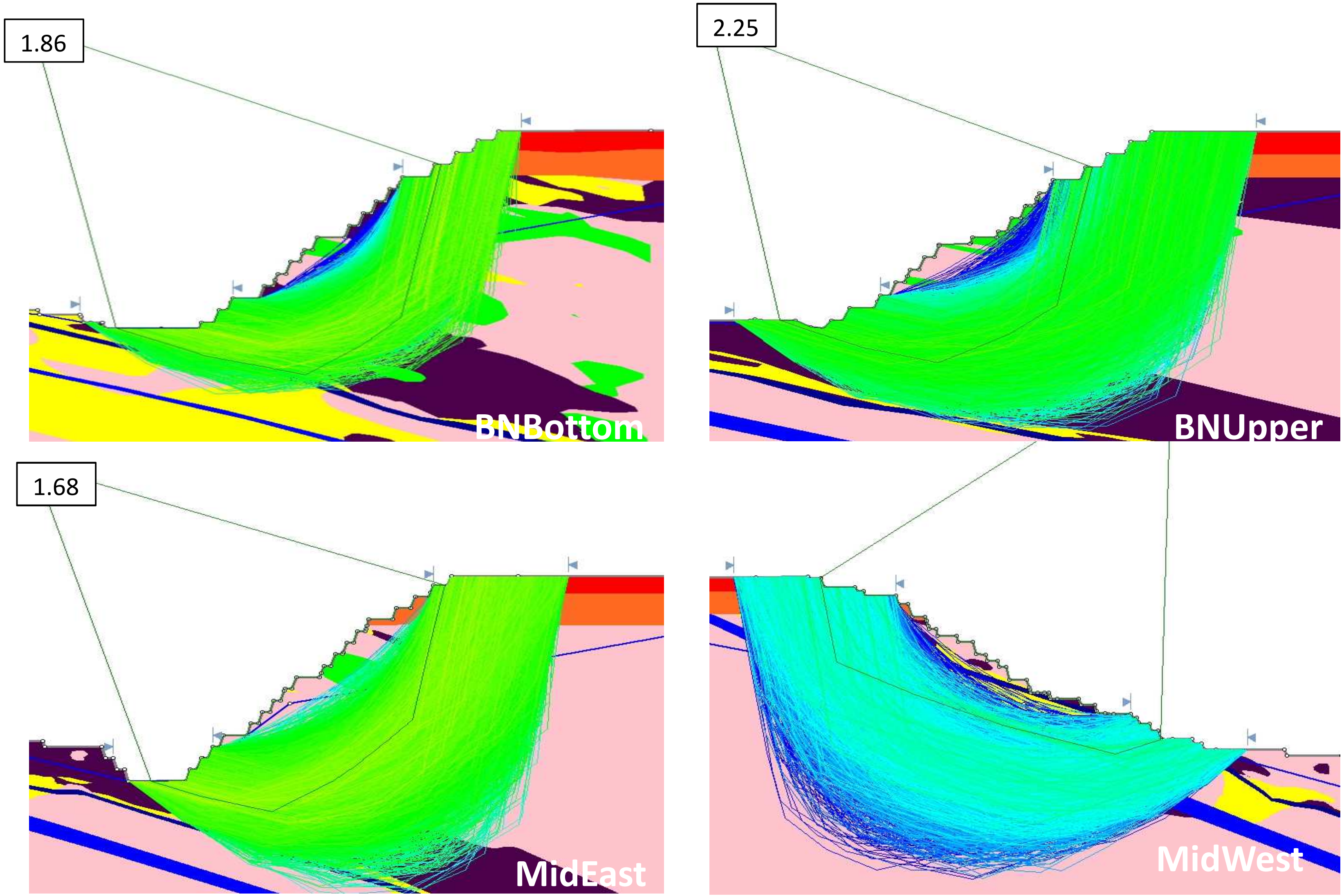
FAILURE INFLUENCED BY DISCONTINUITIES

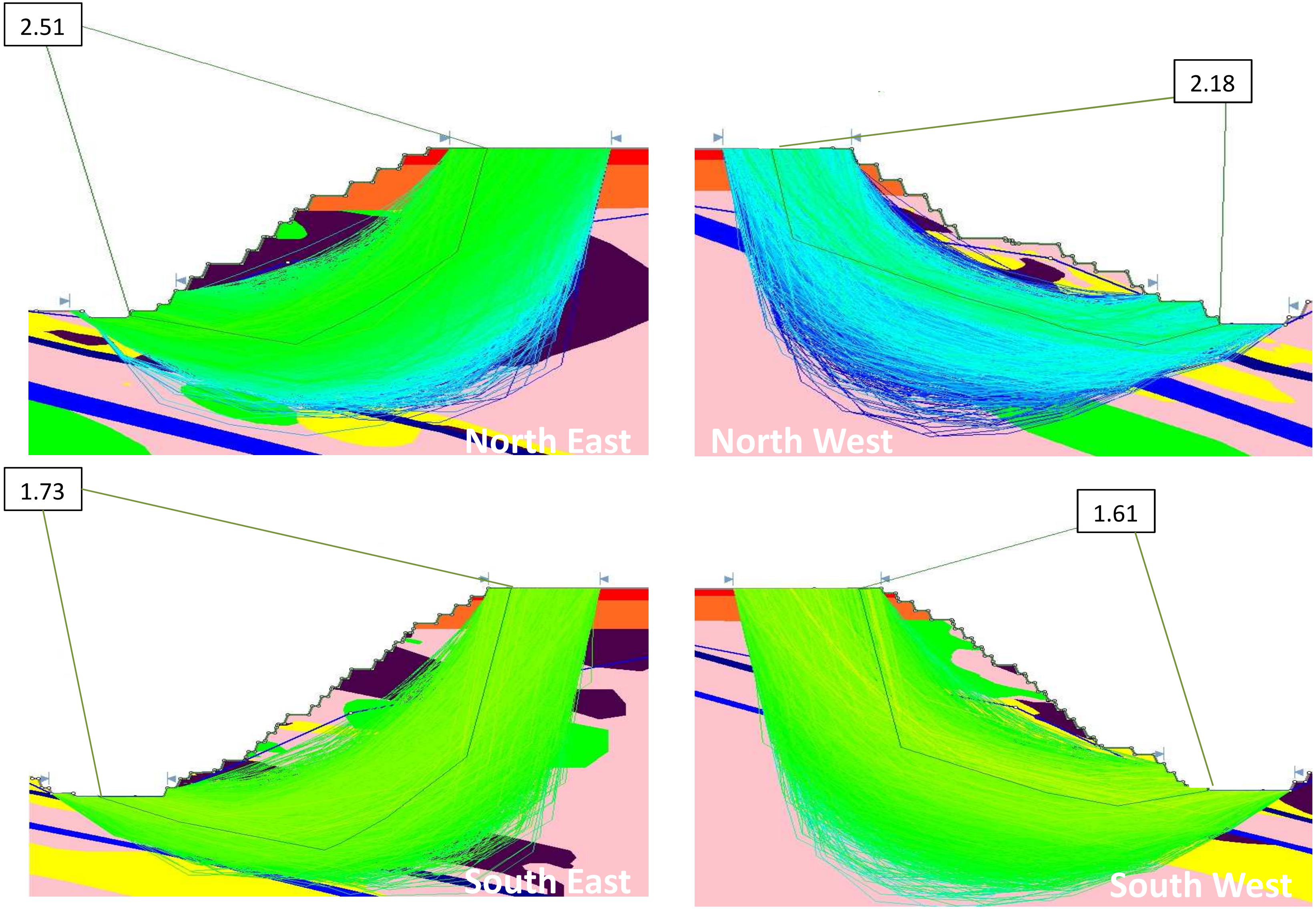
TYPE CODE	DESCRIPTION OF SUB CODES	
	A	B
	PARTIAL FAILURE ON DISCONTINUITY	FAILURE COMPLETELY ON DISCONTINUITY
1	AT 0-10° TO AXIS	AT 0-10° TO AXIS
2	AT 11-20° TO AXIS	AT 11-20° TO AXIS
3	AT 21-30° TO AXIS	AT 21-30° TO AXIS
4	AT 31-40° TO AXIS	AT 31-40° TO AXIS
5	AT 41-50° TO AXIS	AT 41-50° TO AXIS
6	AT 51-70° TO AXIS	AT 51-70° TO AXIS
7	AT 71-90° TO AXIS	AT 71-90° TO AXIS
0	Multiple Discontinuities	Multiple Discontinuities

<u>Example</u>: Failure Type3B: Failure completely on a discontinuity with an orientation of between 21°and 30° to the specimen axis.

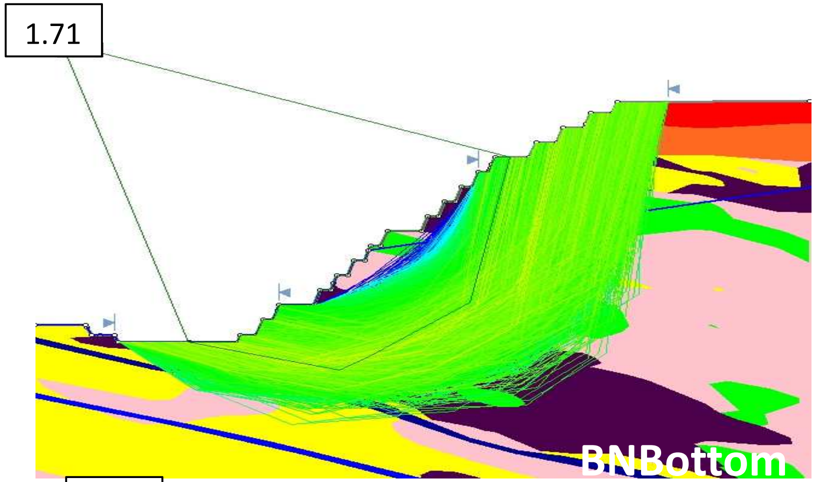


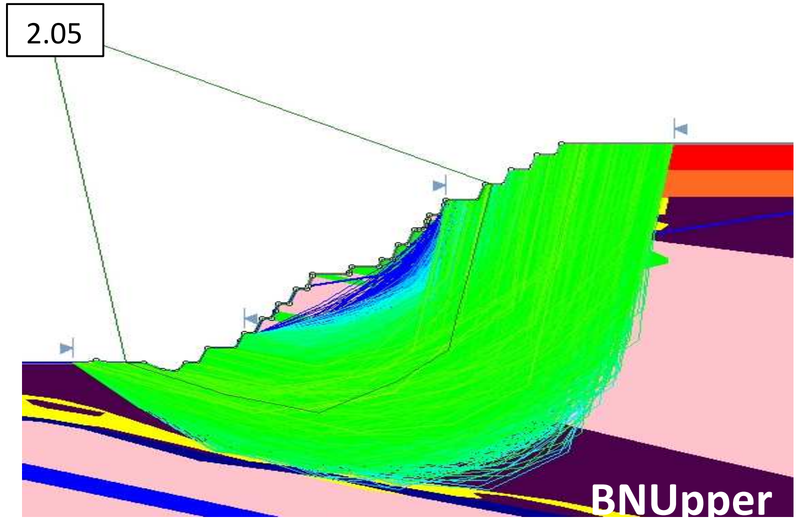
Appendix C: Slide and Phase2 Models

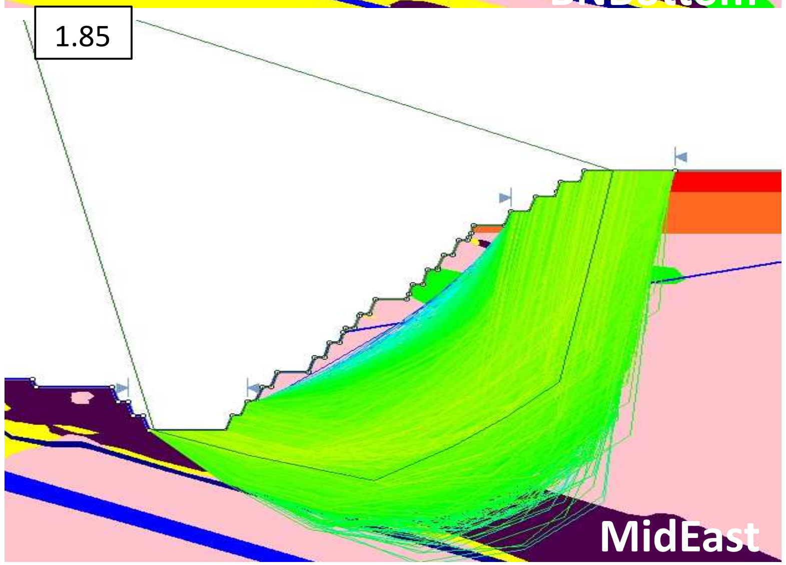

1.86
2.25
1.68
BNBottom
BNUpper
MidEast
MidWest

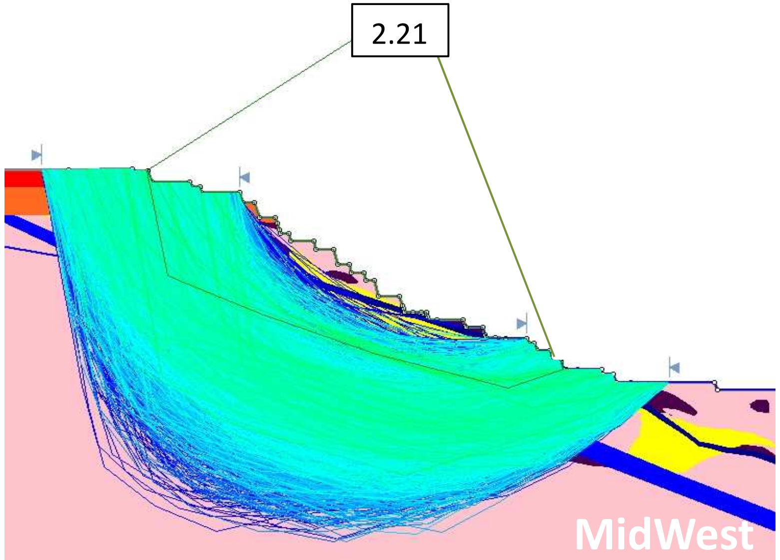


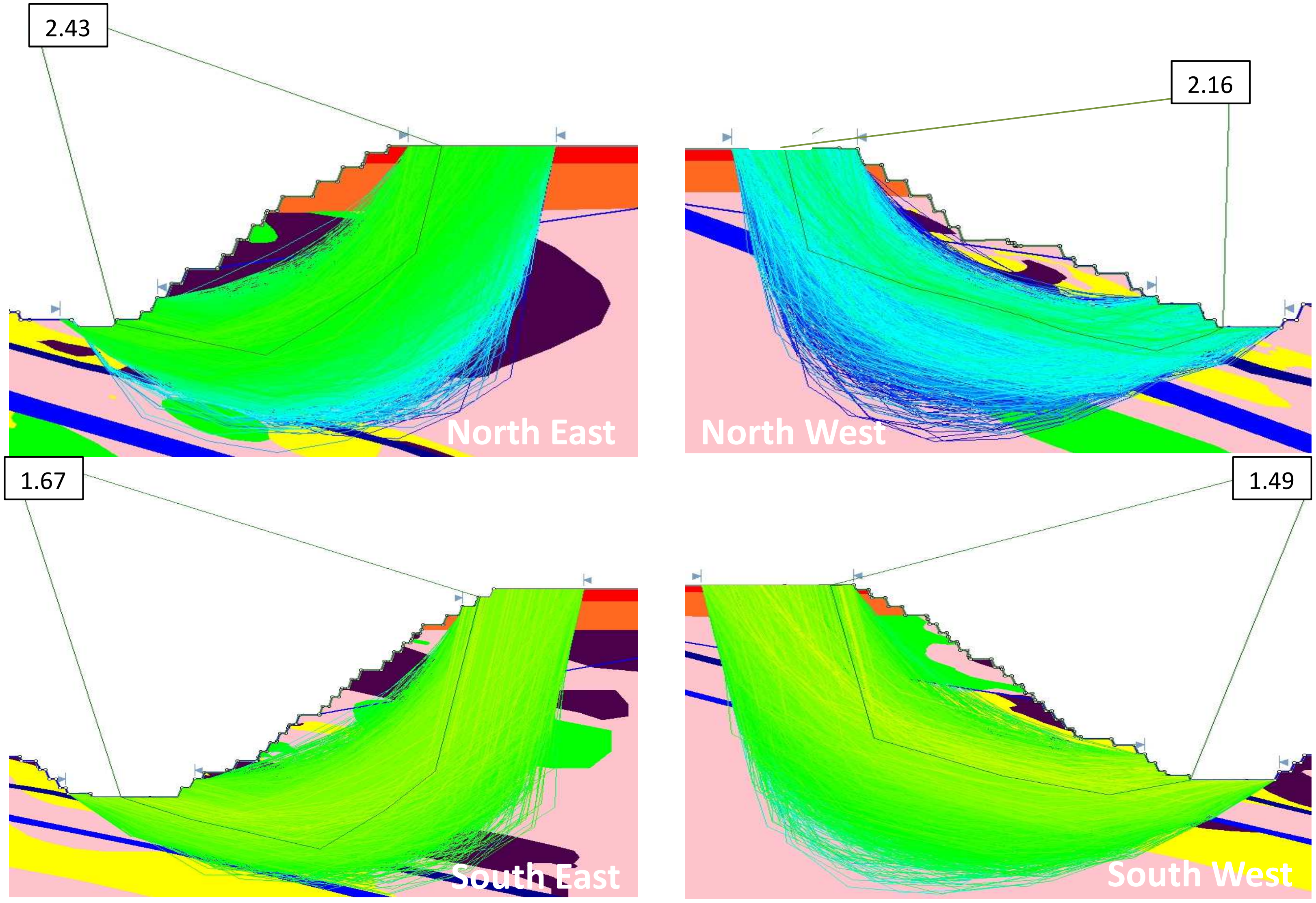
Ohveshlan Pillay	**B2Gold Otjikoto Project** Conservative Models Drained - Slide	Student No.: 706192 Appendix C – Page 2

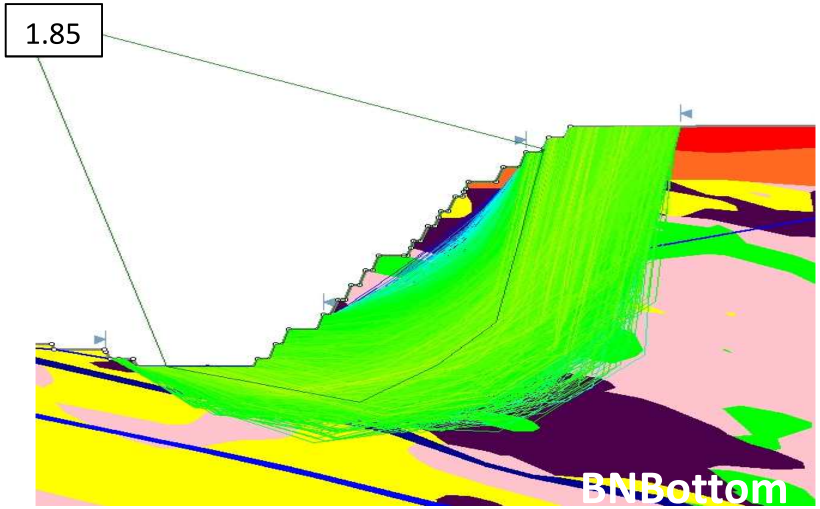

1.71
BNBottom

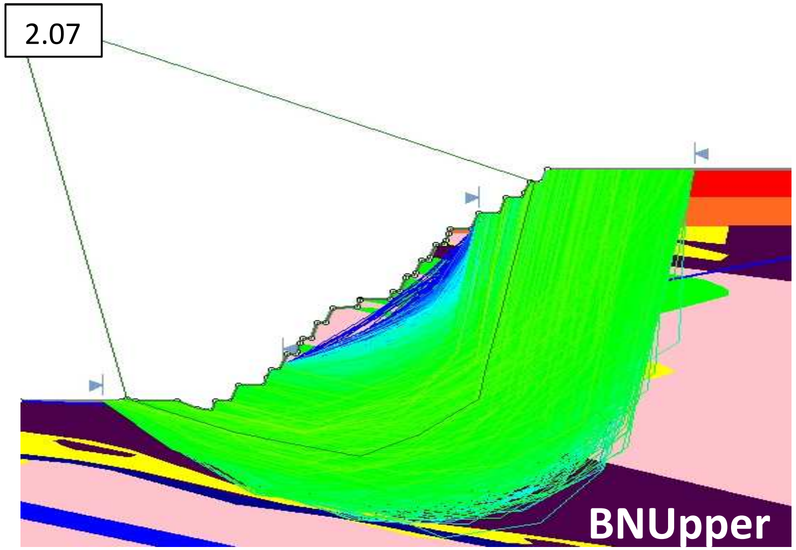

2.05
BNUpper


1.85
MidEast


2.21
MidWest


2.43
2.16
North East
North West
1.67
1.49
South East
South West


1.85
BNBottom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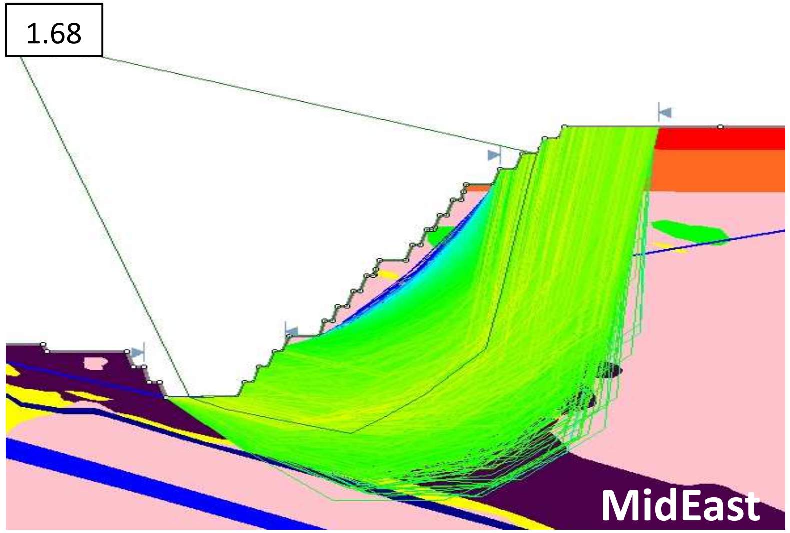
BNUpper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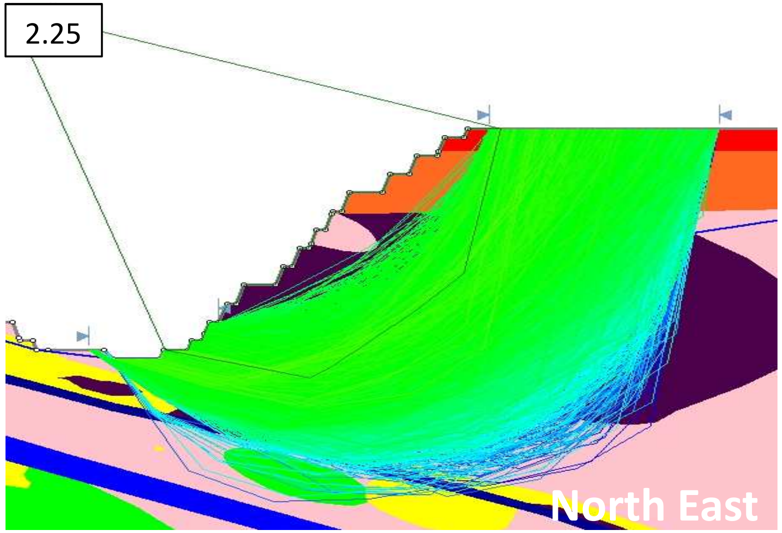
MidEast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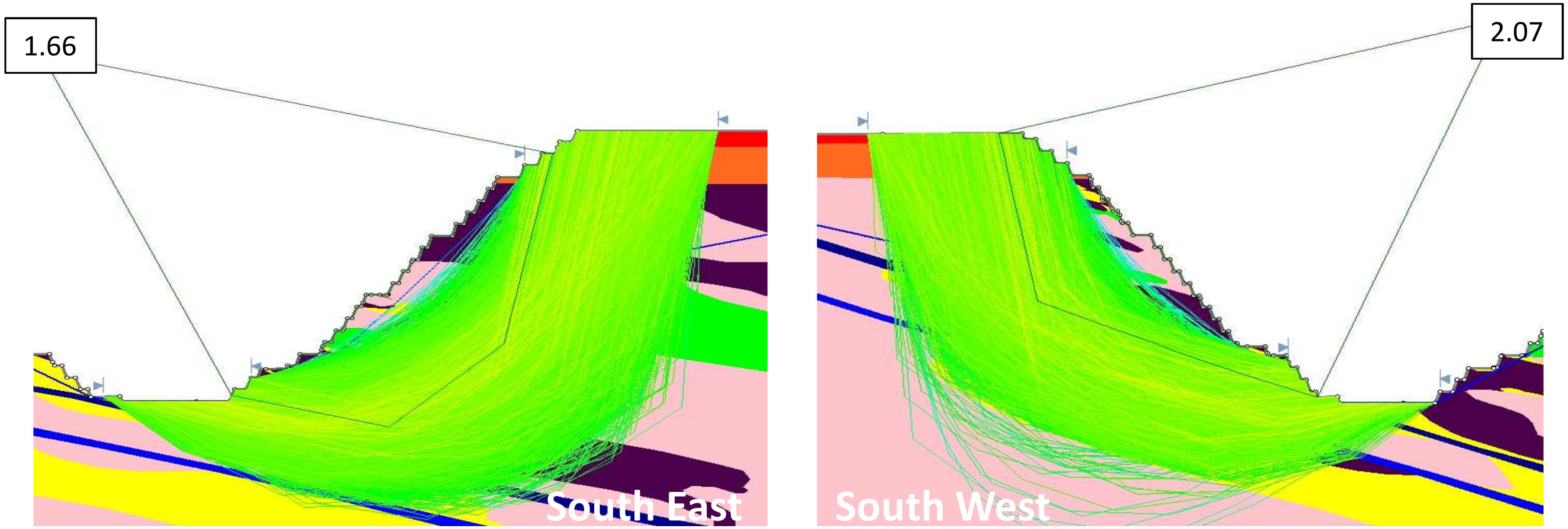
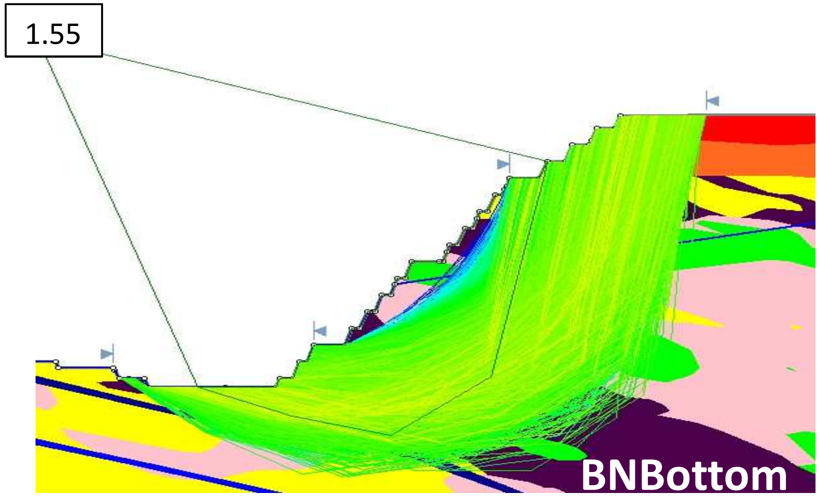
North East


1.66
2.07
South East
South West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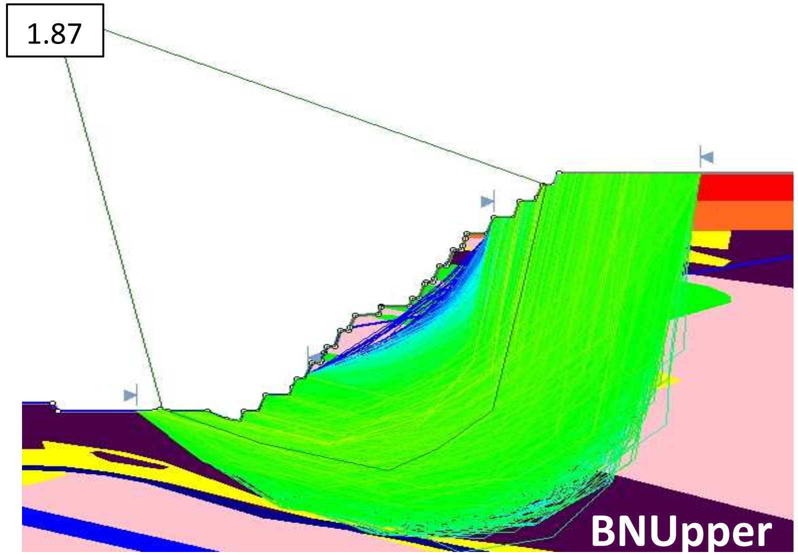
BNBottom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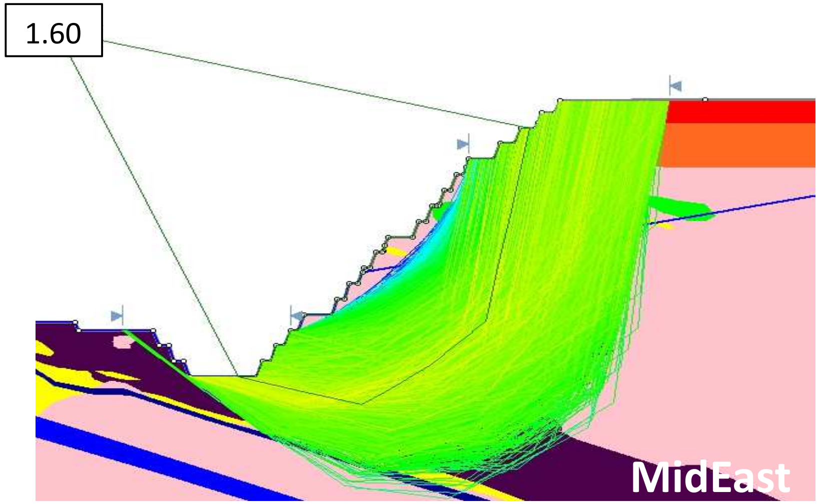
BNUpper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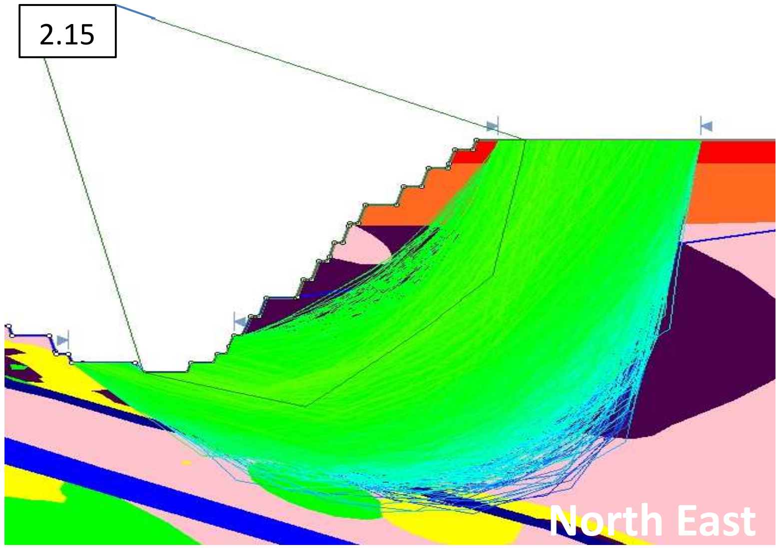
MidEast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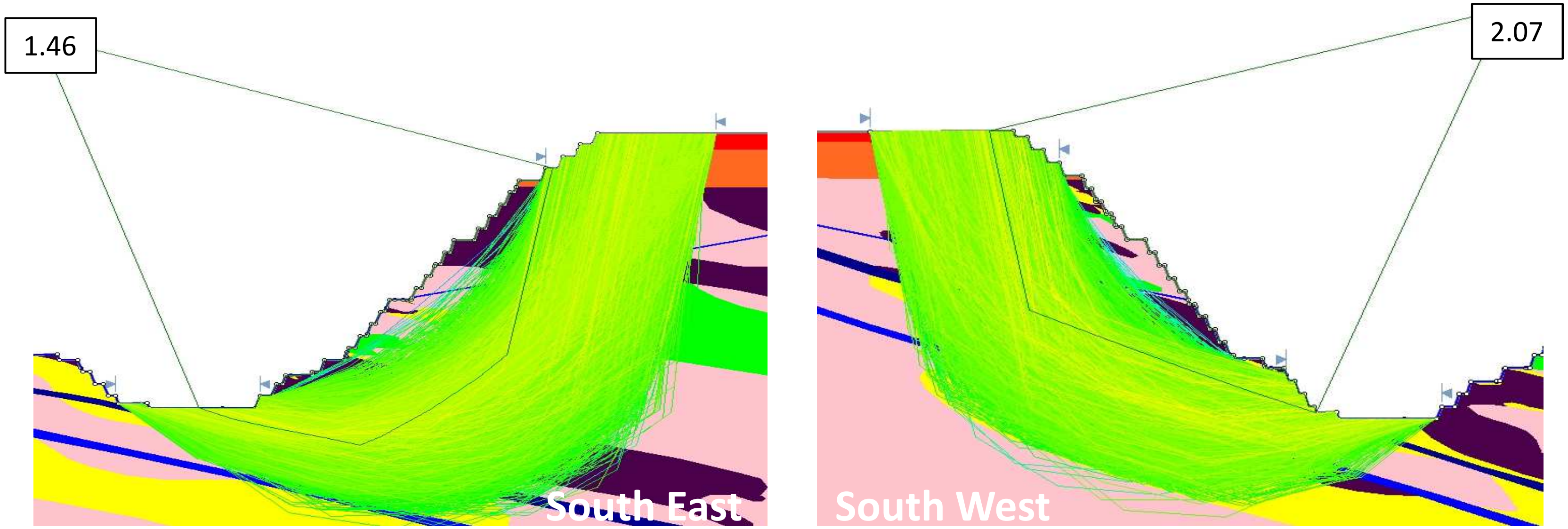
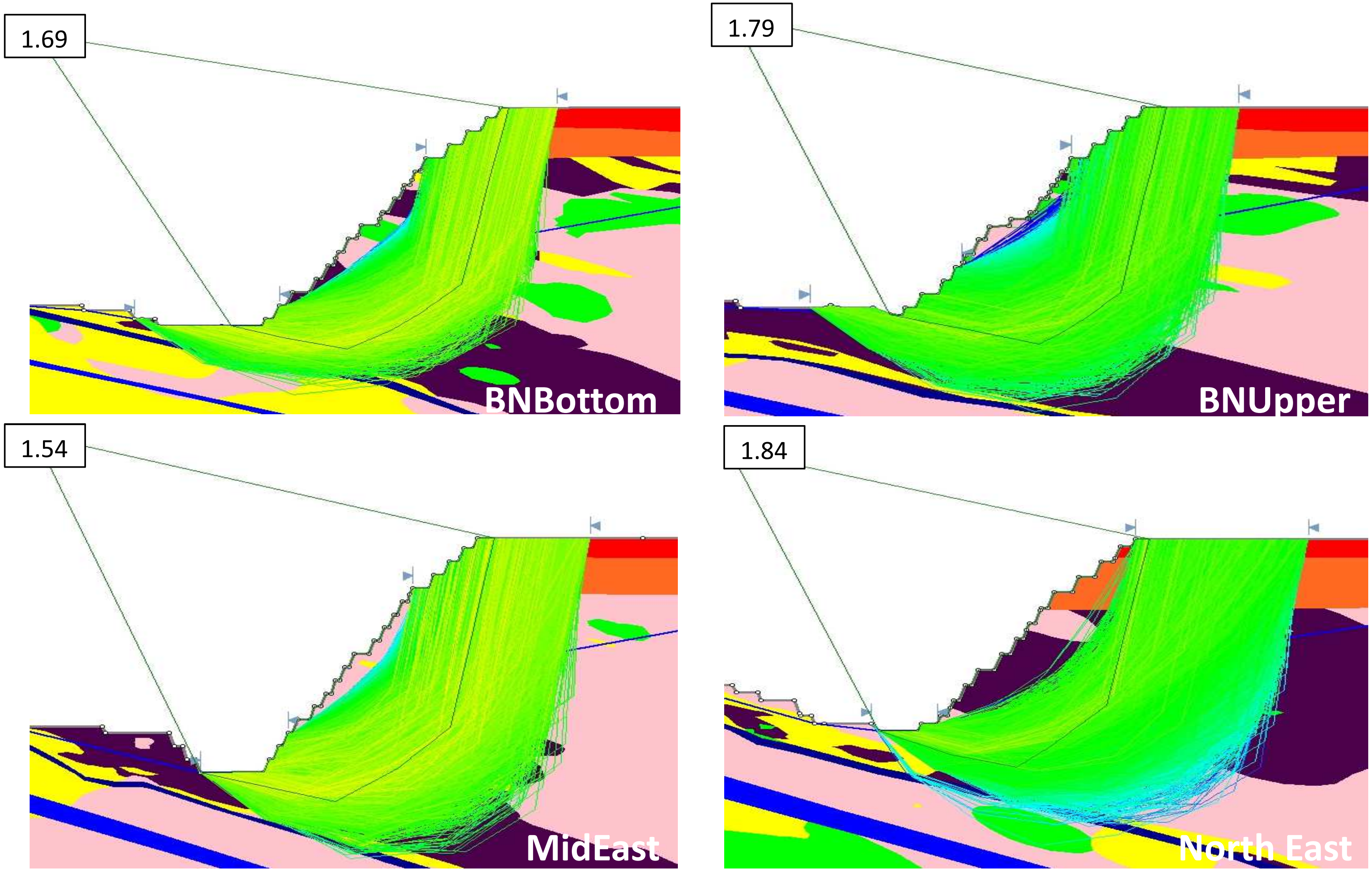
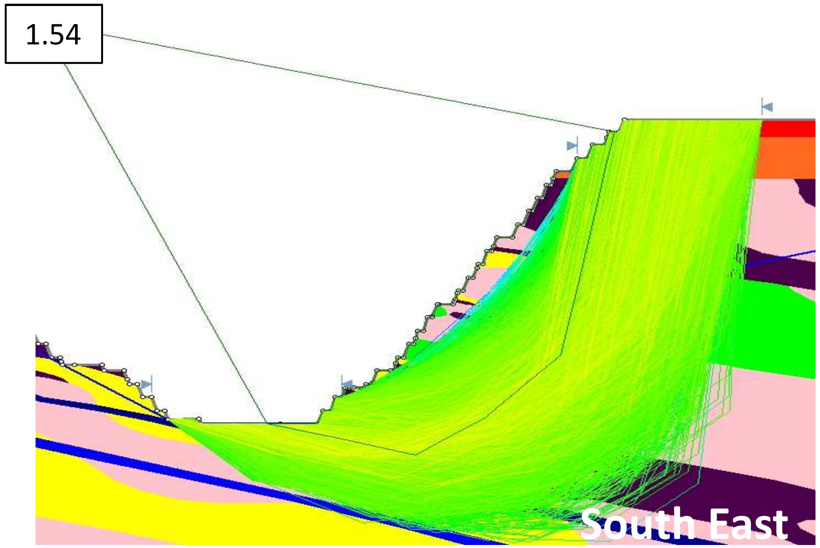
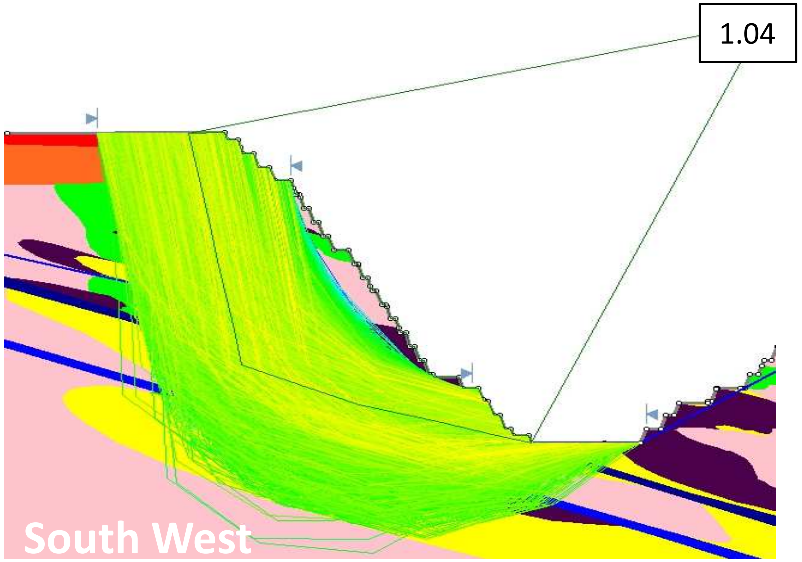
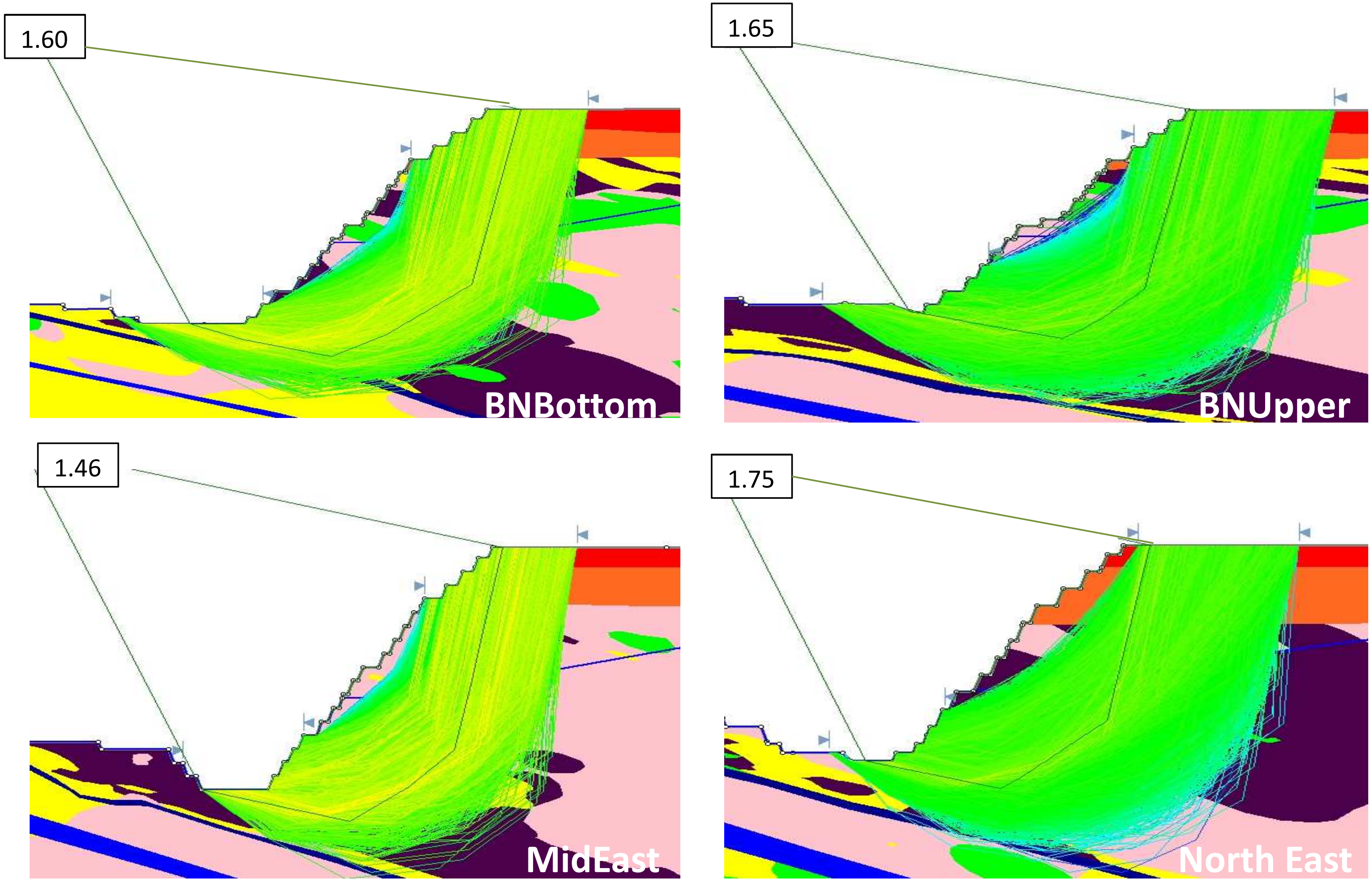
North East

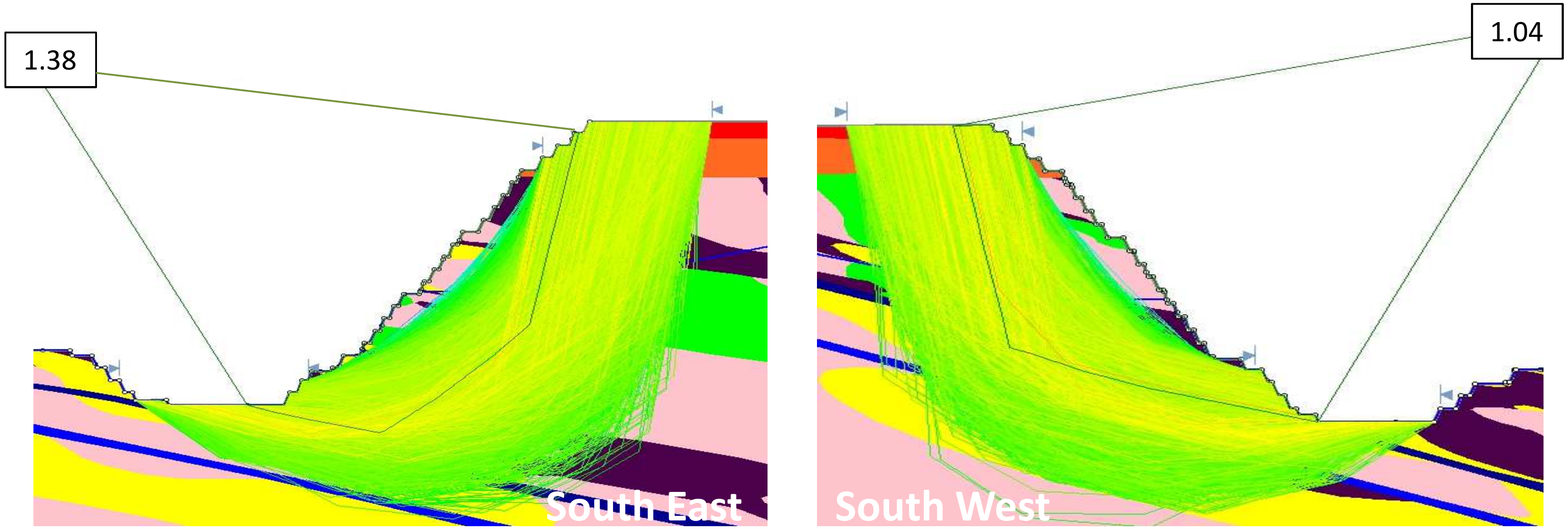

1.46
2.07
South East
South West


1.69
BNBottom
1.79
BNUpper
1.54
MidEast
1.84
North East


1.54
South East


1.04
South West


1.60
BNBottom
1.65
BNUpper
1.46
MidEast
1.75
North East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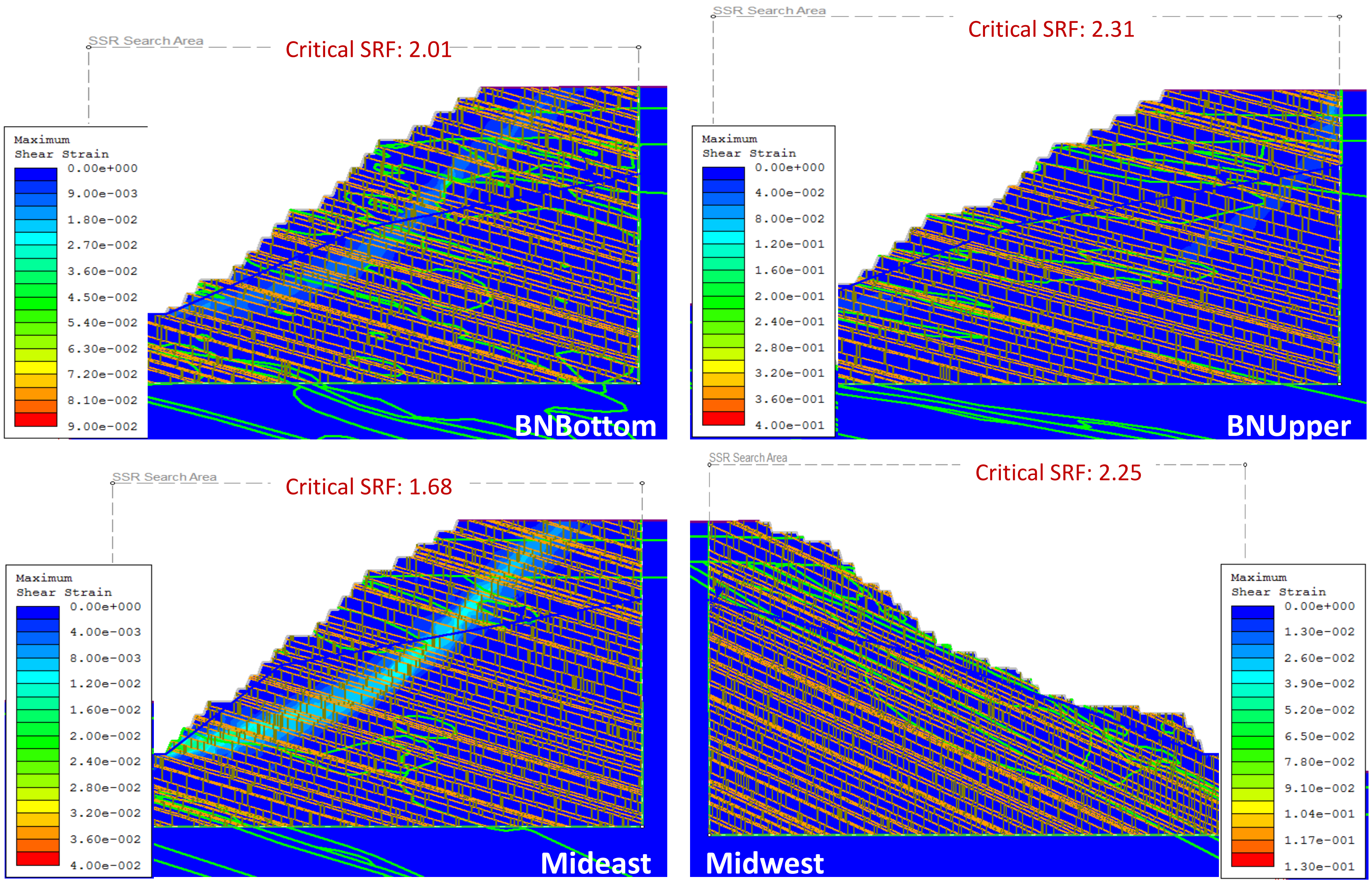
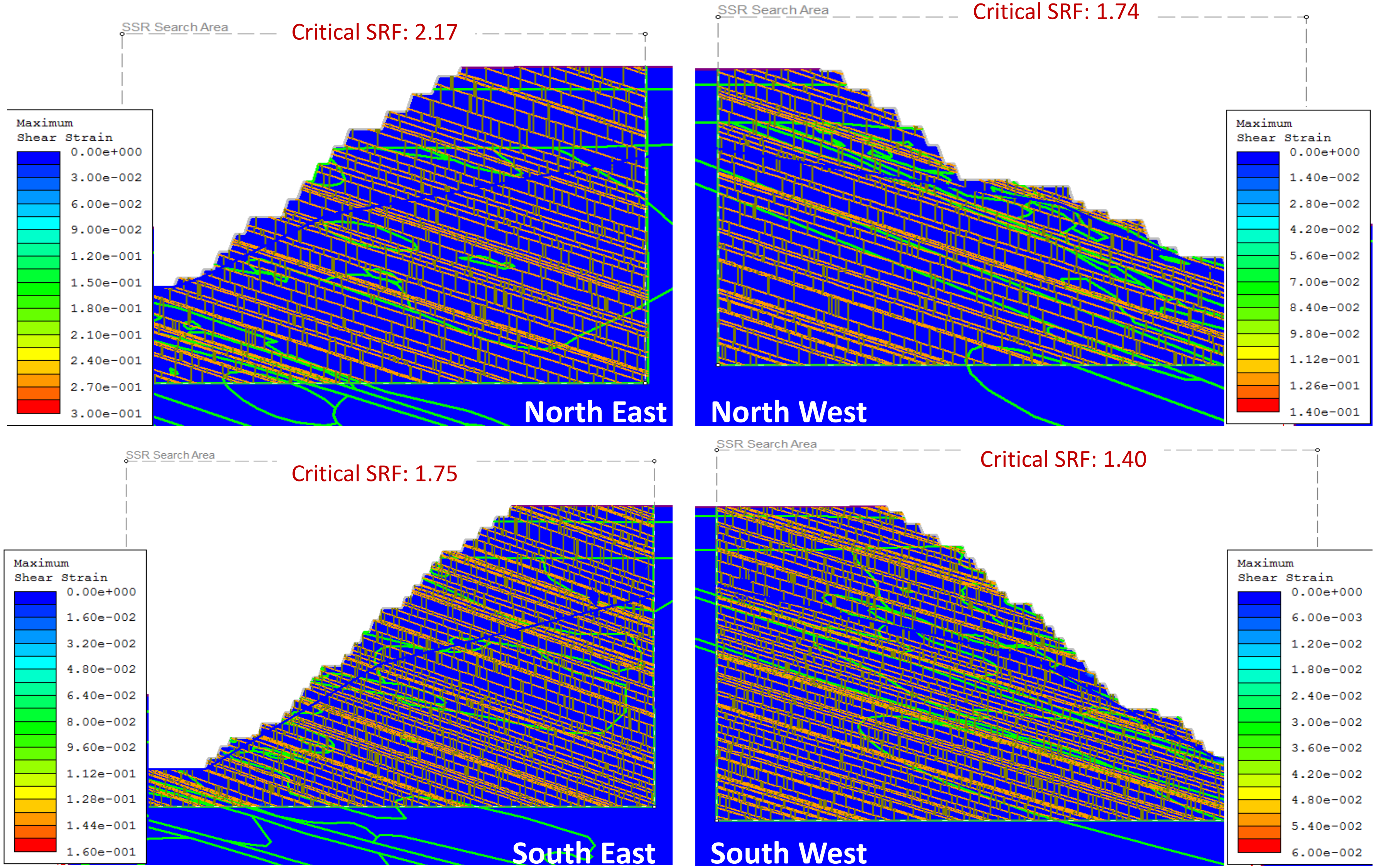
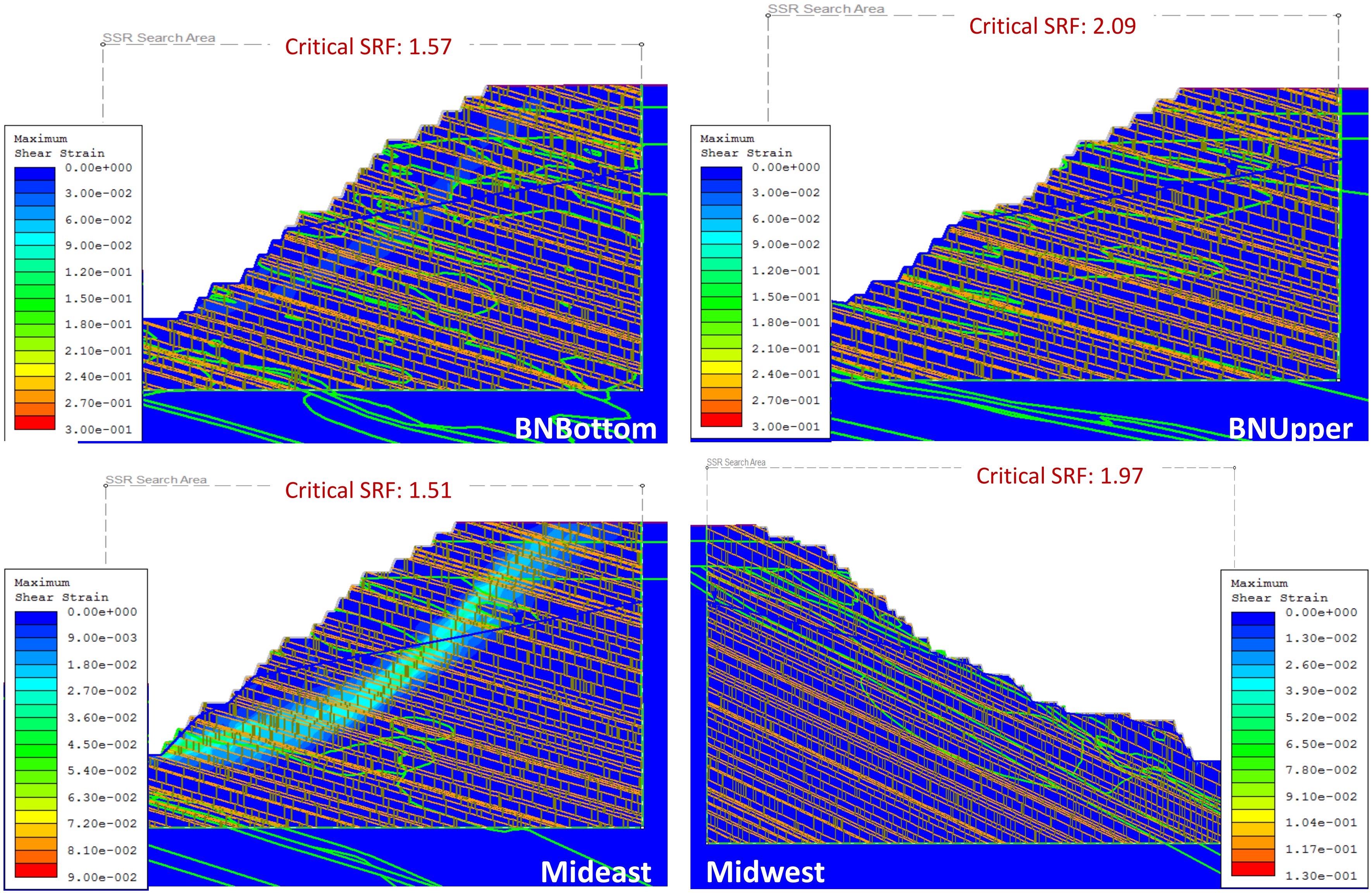
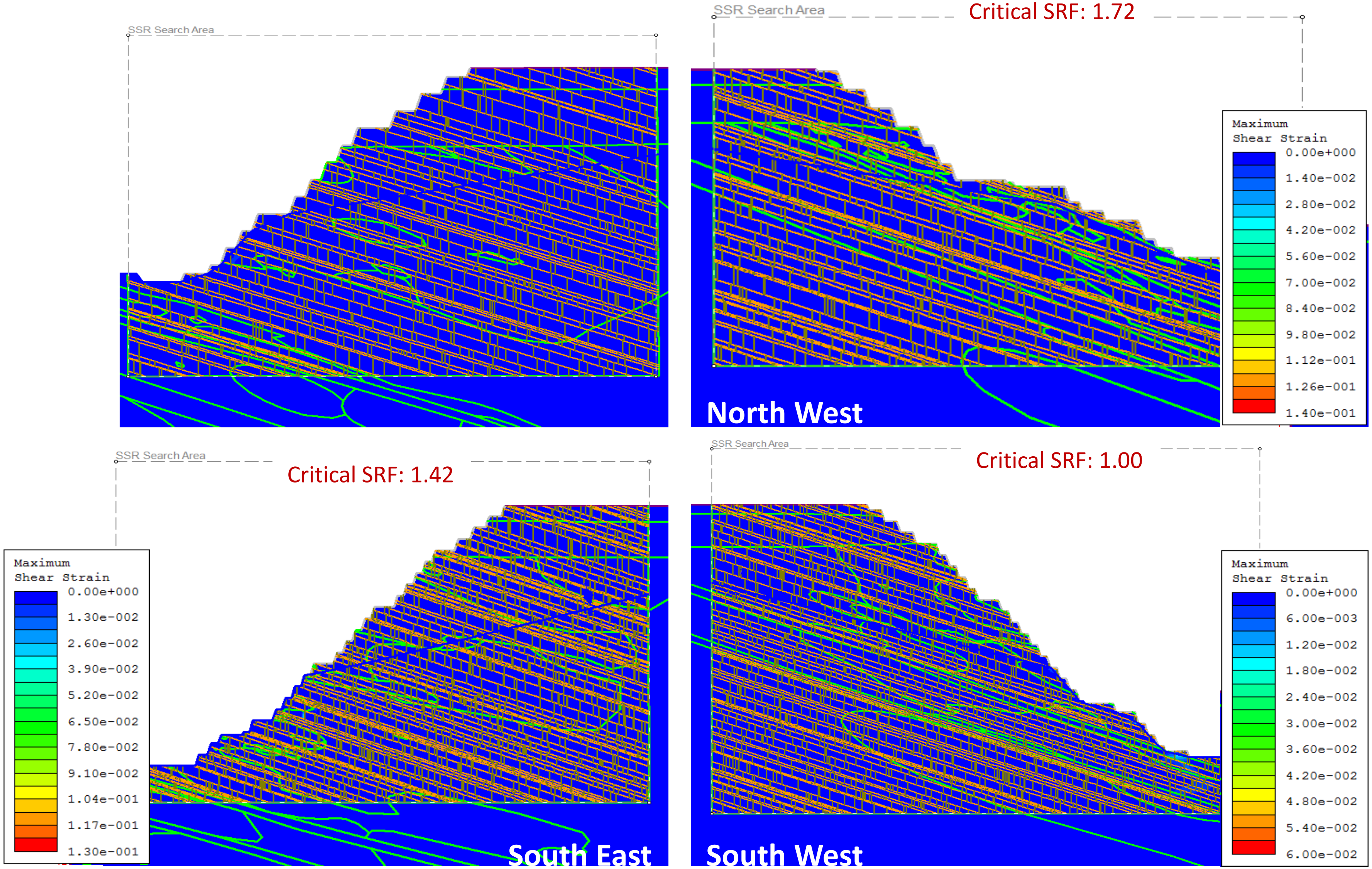
1.04
South East
South West


SSR Search Area
Critical SRF: 2.01
Maximum Shear Strain
0.00e+000
9.00e-003
1.80e-002
2.70e-002
3.60e-002
4.50e-002
5.40e-002
6.30e-002
7.20e-002
8.10e-002
9.00e-002
BNBottom
SSR Search Area
Critical SRF: 2.31
Maximum Shear Strain
0.00e+000
4.00e-002
8.00e-002
1.20e-001
1.60e-001
2.00e-001
2.40e-001
2.80e-001
3.20e-001
3.60e-001
4.00e-001
BNUpper
SSR Search Area
Critical SRF: 1.68
Maximum Shear Strain
0.00e+000
4.00e-003
8.00e-003
1.20e-002
1.60e-002
2.00e-002
2.40e-002
2.80e-002
3.20e-002
3.60e-002
4.00e-002
Mideast
SSR Search Area
Critical SRF: 2.25
Maximum Shear Strain
0.00e+000
1.30e-002
2.60e-002
3.90e-002
5.20e-002
6.50e-002
7.80e-002
9.10e-002
1.04e-001
1.17e-001
1.30e-001
Midwest

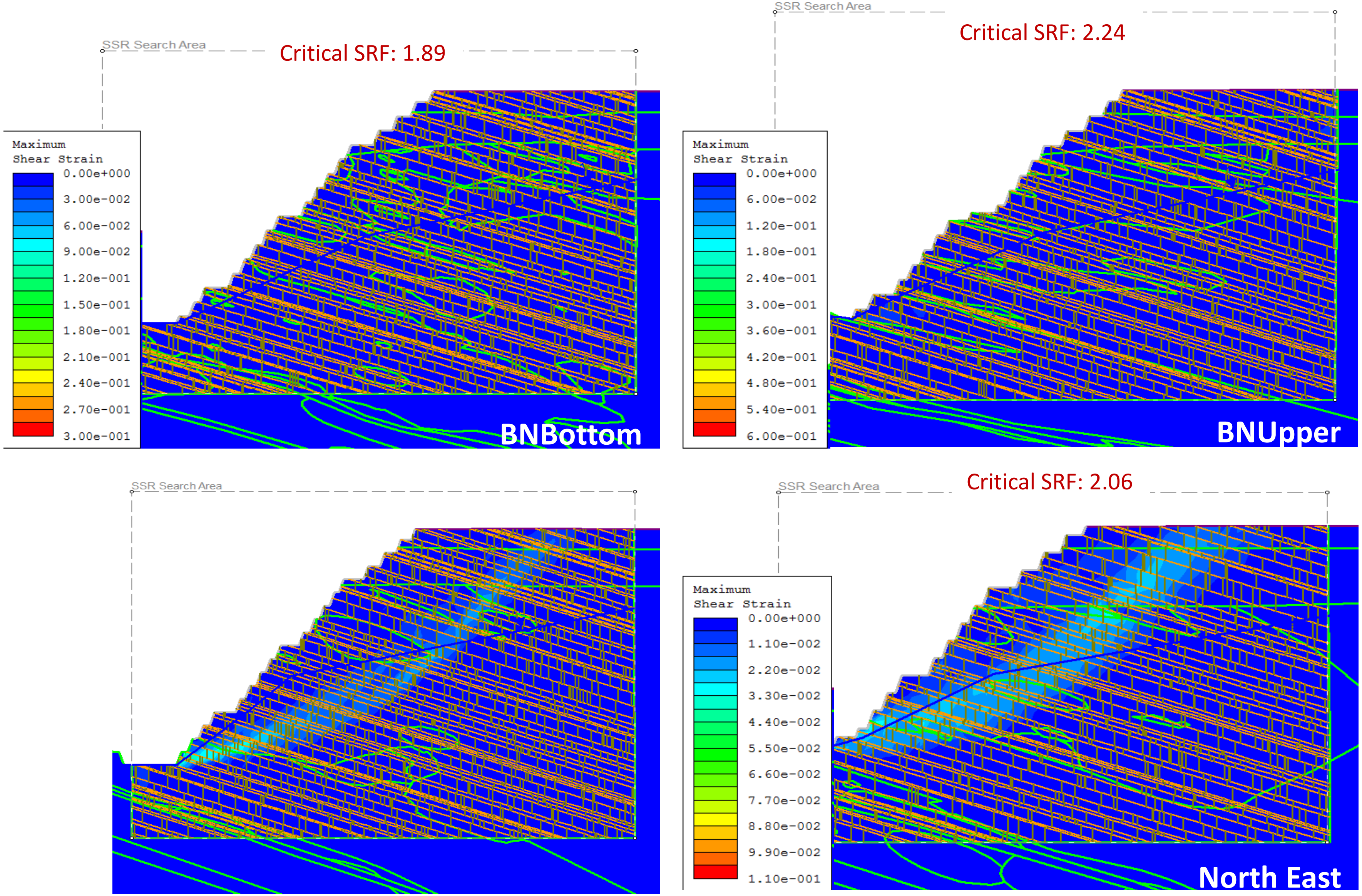


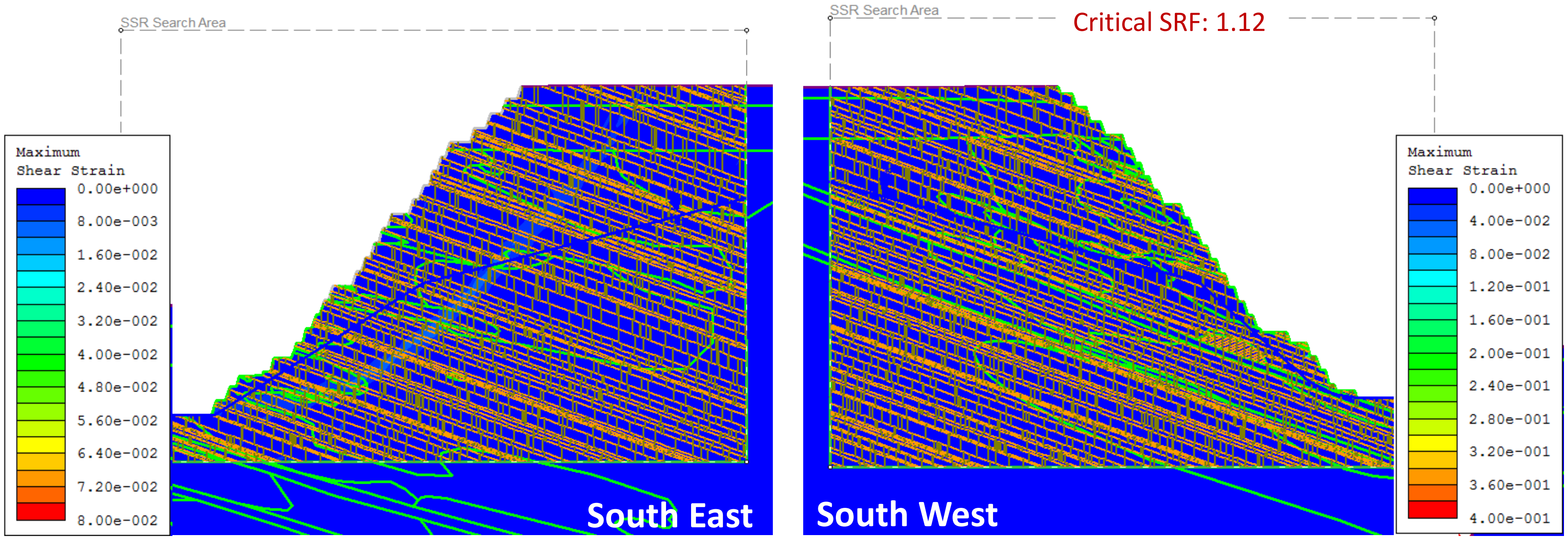
SSR Search Area
Critical SRF: 2.17
Maximum Shear Strain
0.00e+000
3.00e-002
6.00e-002
9.00e-002
1.20e-001
1.50e-001
1.80e-001
2.10e-001
2.40e-001
2.70e-001
3.00e-001
North East
SSR Search Area
Critical SRF: 1.74
Maximum Shear Strain
0.00e+000
1.40e-002
2.80e-002
4.20e-002
5.60e-002
7.00e-002
8.40e-002
9.80e-002
1.12e-001
1.26e-001
1.40e-001
North West
SSR Search Area
Critical SRF: 1.75
Maximum Shear Strain
0.00e+000
1.60e-002
3.20e-002
4.80e-002
6.40e-002
8.00e-002
9.60e-002
1.12e-001
1.28e-001
1.44e-001
1.60e-001
South East
SSR Search Area
Critical SRF: 1.40
Maximum Shear Strain
0.00e+000
6.00e-003
1.20e-002
1.80e-002
2.40e-002
3.00e-002
3.60e-002
4.20e-002
4.80e-002
5.40e-002
6.00e-002
South West

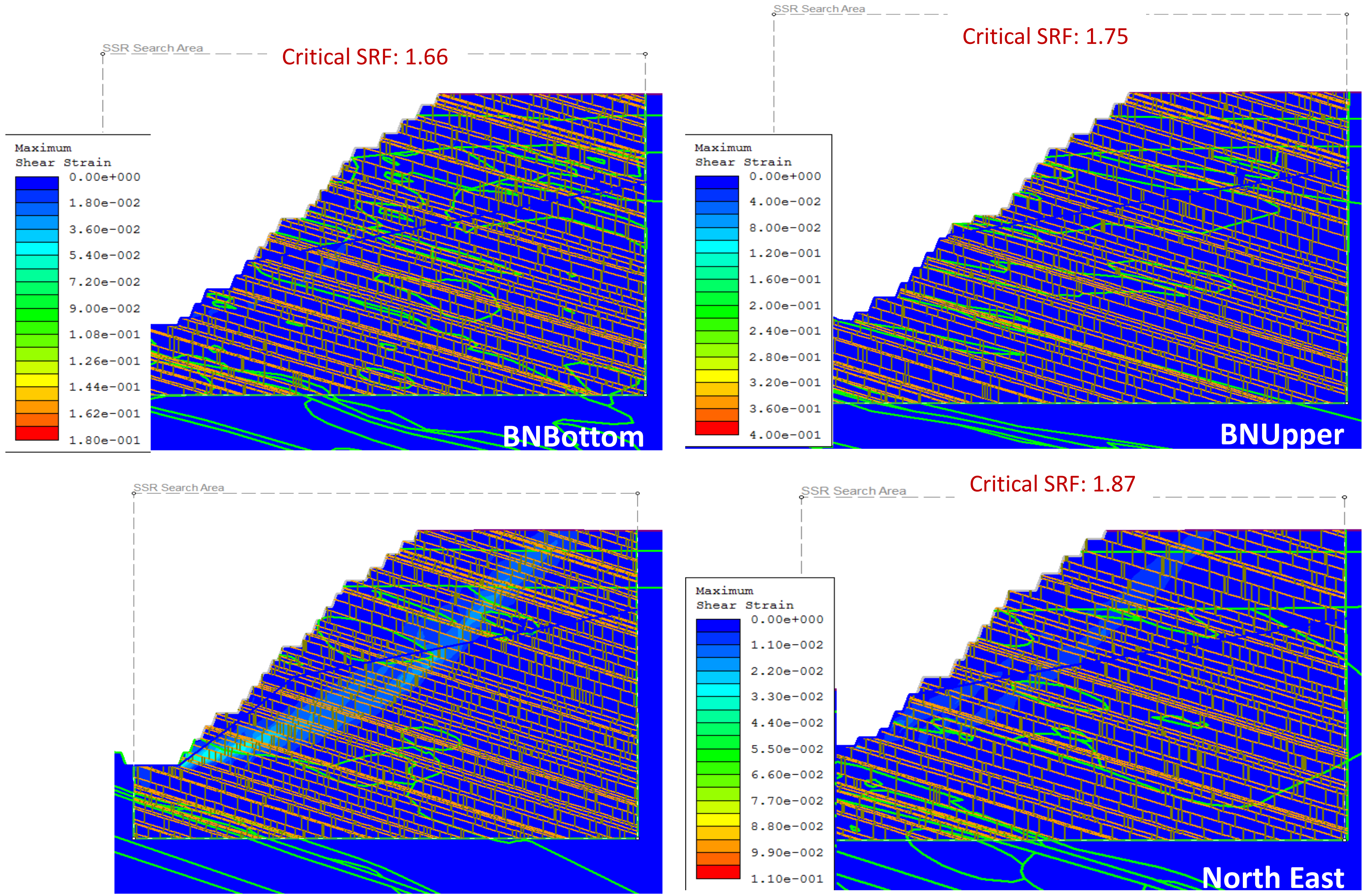

SSR Search Area
Critical SRF: 1.57
Maximum
Shear Strain
0.00e+000
3.00e-002
6.00e-002
9.00e-002
1.20e-001
1.50e-001
1.80e-001
2.10e-001
2.40e-001
2.70e-001
3.00e-001
BNBottom
SSR Search Area
Critical SRF: 2.09
Maximum
Shear Strain
0.00e+000
3.00e-002
6.00e-002
9.00e-002
1.20e-001
1.50e-001
1.80e-001
2.10e-001
2.40e-001
2.70e-001
3.00e-001
BNUpper
SSR Search Area
Critical SRF: 1.51
Maximum
Shear Strain
0.00e+000
9.00e-003
1.80e-002
2.70e-002
3.60e-002
4.50e-002
5.40e-002
6.30e-002
7.20e-002
8.10e-002
9.00e-002
Mideast
SSR Search Area
Critical SRF: 1.97
Maximum
Shear Strain
0.00e+000
1.30e-002
2.60e-002
3.90e-002
5.20e-002
6.50e-002
7.80e-002
9.10e-002
1.04e-001
1.17e-001
1.30e-001
Midwest



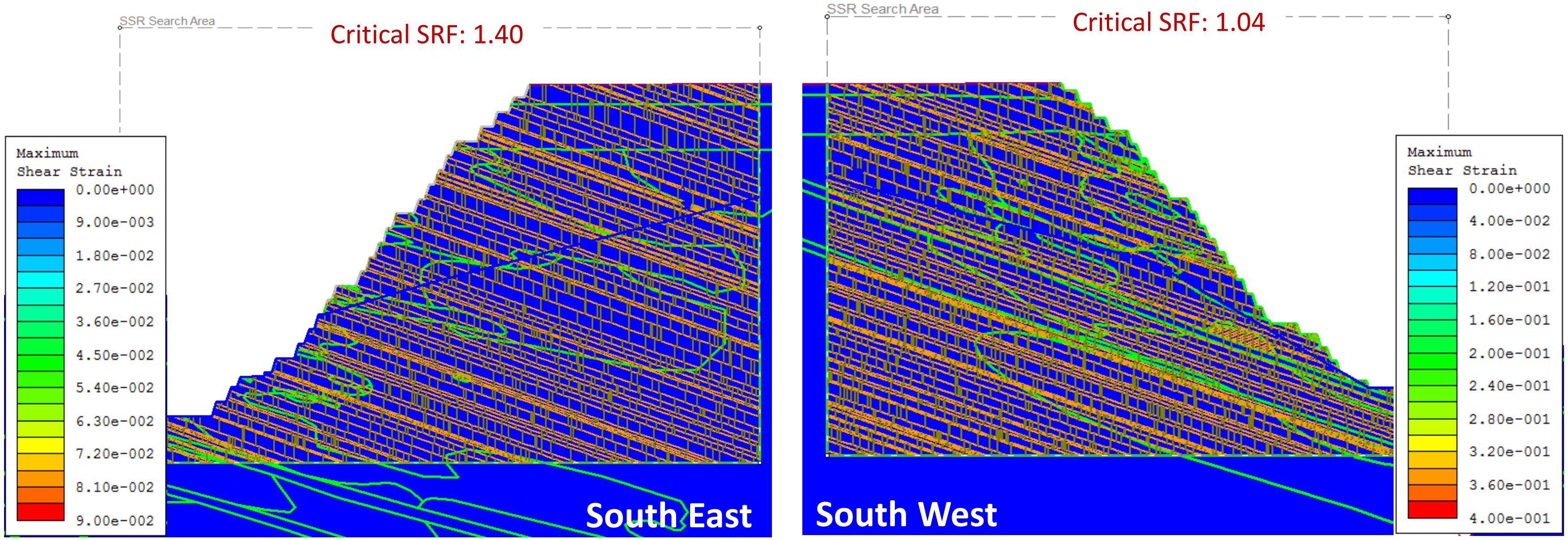
SSR Search Area
SSR Search Area
Critical SRF: 1.72
Maximum
Shear Strain
0.00e+000
1.40e-002
2.80e-002
4.20e-002
5.60e-002
7.00e-002
8.40e-002
9.80e-002
1.12e-001
1.26e-001
1.40e-001
North West
SSR Search Area
Critical SRF: 1.42
Maximum
Shear Strain
0.00e+000
1.30e-002
2.60e-002
3.90e-002
5.20e-002
6.50e-002
7.80e-002
9.10e-002
1.04e-001
1.17e-001
1.30e-001
South East
SSR Search Area
Critical SRF: 1.00
Maximum
Shear Strain
0.00e+000
6.00e-003
1.20e-002
1.80e-002
2.40e-002
3.00e-002
3.60e-002
4.20e-002
4.80e-002
5.40e-002
6.00e-002
South West


SSR Search Area
Critical SRF: 1.89
Maximum
Shear Strain
0.00e+000
3.00e-002
6.00e-002
9.00e-002
1.20e-001
1.50e-001
1.80e-001
2.10e-001
2.40e-001
2.70e-001
3.00e-001
BNBottom
SSR Search Area
Critical SRF: 2.24
Maximum
Shear Strain
0.00e+000
6.00e-002
1.20e-001
1.80e-001
2.40e-001
3.00e-001
3.60e-001
4.20e-001
4.80e-001
5.40e-001
6.00e-001
BNUpper
SSR Search Area
SSR Search Area
Critical SRF: 2.06
Maximum
Shear Strain
0.00e+000
1.10e-002
2.20e-002
3.30e-002
4.40e-002
5.50e-002
6.60e-002
7.70e-002
8.80e-002
9.90e-002
1.10e-001
North East

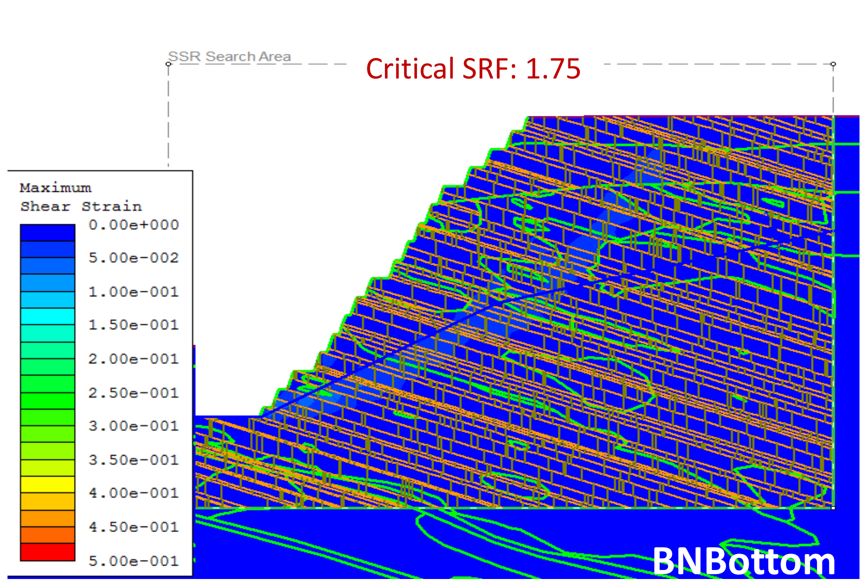

SSR Search Area
Maximum
Shear Strain
0.00e+000
8.00e-003
1.60e-002
2.40e-002
3.20e-002
4.00e-002
4.80e-002
5.60e-002
6.40e-002
7.20e-002
8.00e-002
South East
SSR Search Area
Critical SRF: 1.12
South West
Maximum
Shear Strain
0.00e+000
4.00e-002
8.00e-002
1.20e-001
1.60e-001
2.00e-001
2.40e-001
2.80e-001
3.20e-001
3.60e-001
4.00e-001

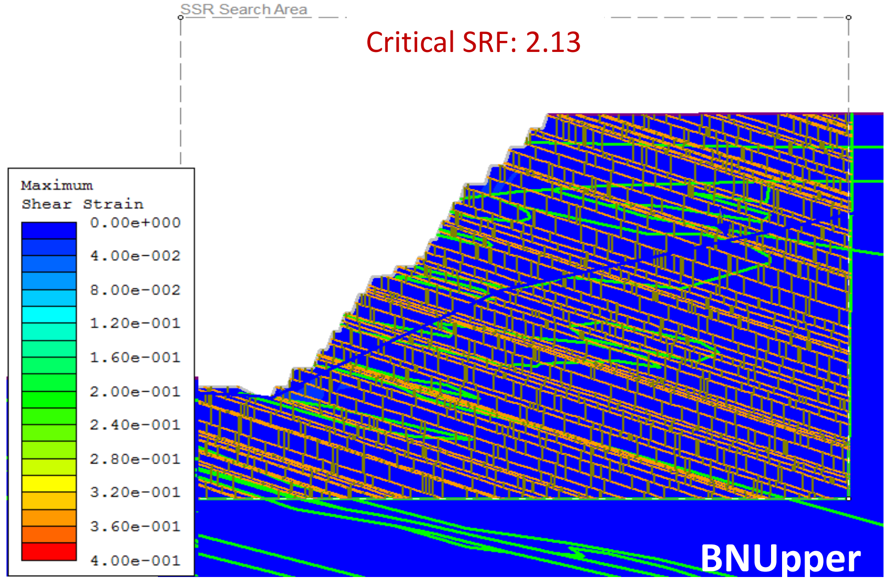


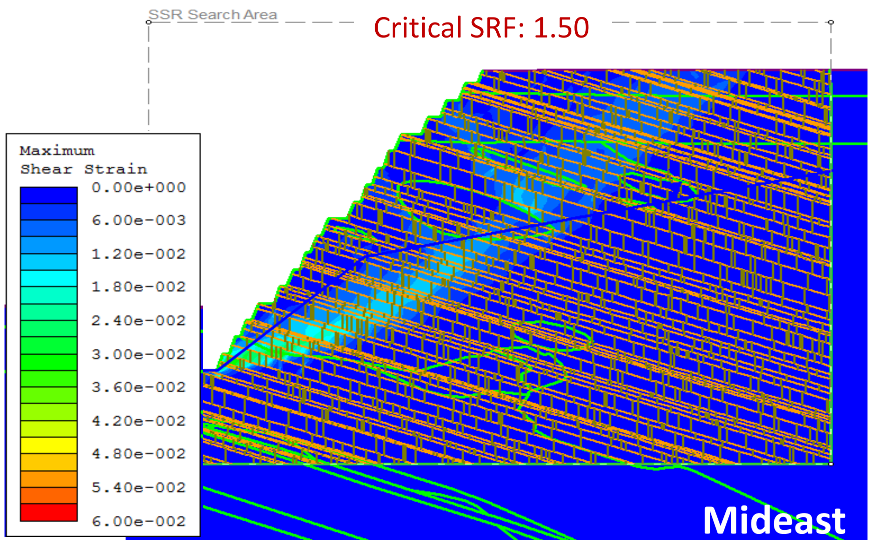
SSR Search Area
Critical SRF: 1.66
Maximum
Shear Strain
0.00e+000
1.80e-002
3.60e-002
5.40e-002
7.20e-002
9.00e-002
1.08e-001
1.26e-001
1.44e-001
1.62e-001
1.80e-001
BNBottom
SSR Search Area
Critical SRF: 1.75
Maximum
Shear Strain
0.00e+000
4.00e-002
8.00e-002
1.20e-001
1.60e-001
2.00e-001
2.40e-001
2.80e-001
3.20e-001
3.60e-001
4.00e-001
BNUpper
SSR Search Area
SSR Search Area
Critical SRF: 1.87
Maximum
Shear Strain
0.00e+000
1.10e-002
2.20e-002
3.30e-002
4.40e-002
5.50e-002
6.60e-002
7.70e-002
8.80e-002
9.90e-002
1.10e-001
North East

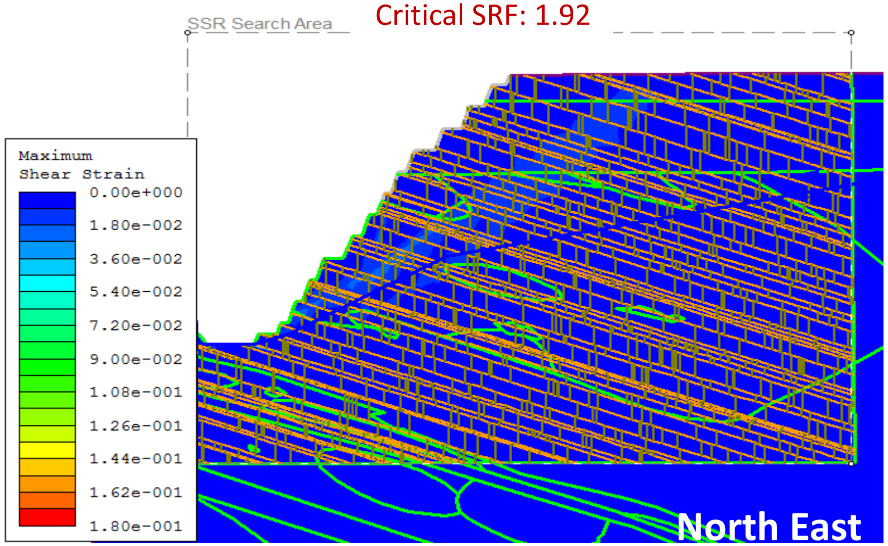


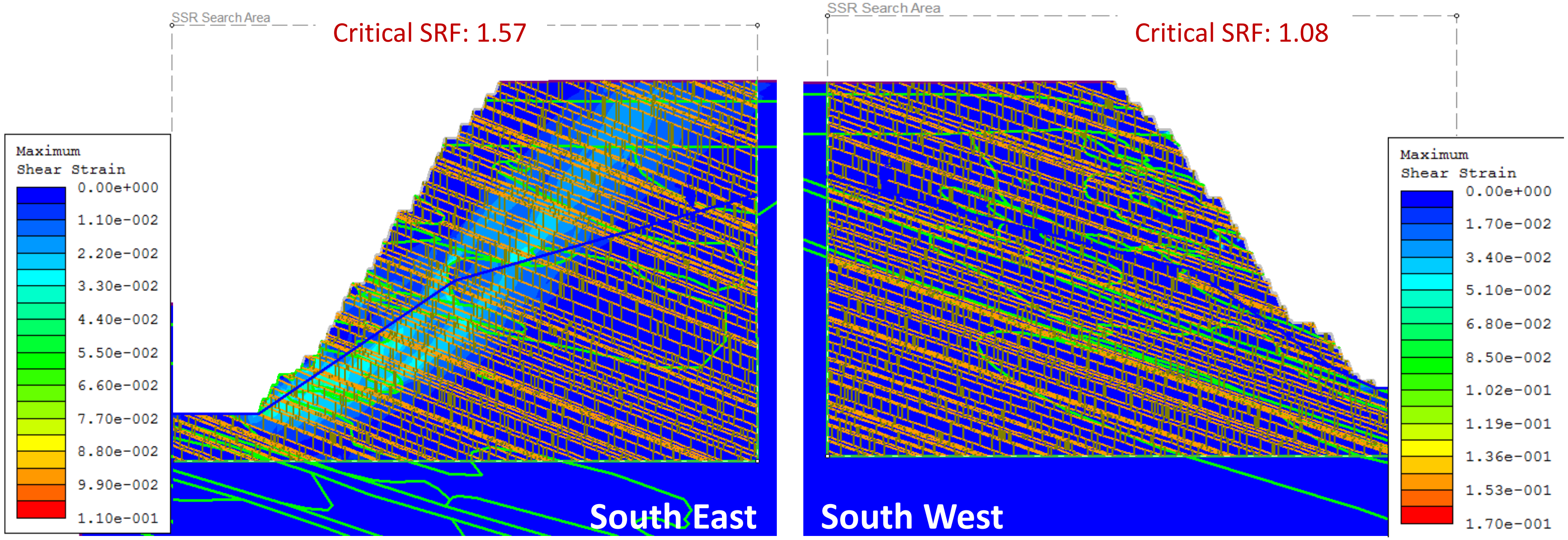
SSR Search Area
Critical SRF: 1.40
Maximum
Shear Strain
0.00e+000
9.00e-003
1.80e-002
2.70e-002
3.60e-002
4.50e-002
5.40e-002
6.30e-002
7.20e-002
8.10e-002
9.00e-002
South East
SSR Search Area
Critical SRF: 1.04
Maximum
Shear Strain
0.00e+000
4.00e-002
8.00e-002
1.20e-001
1.60e-001
2.00e-001
2.40e-001
2.80e-001
3.20e-001
3.60e-001
4.00e-001
South West


SSR Search Area
Critical SRF: 1.75
Maximum
Shear Strain
0.00e+000
5.00e-002
1.00e-001
1.50e-001
2.00e-001
2.50e-001
3.00e-001
3.50e-001
4.00e-001
4.50e-001
5.00e-001
BNBottom

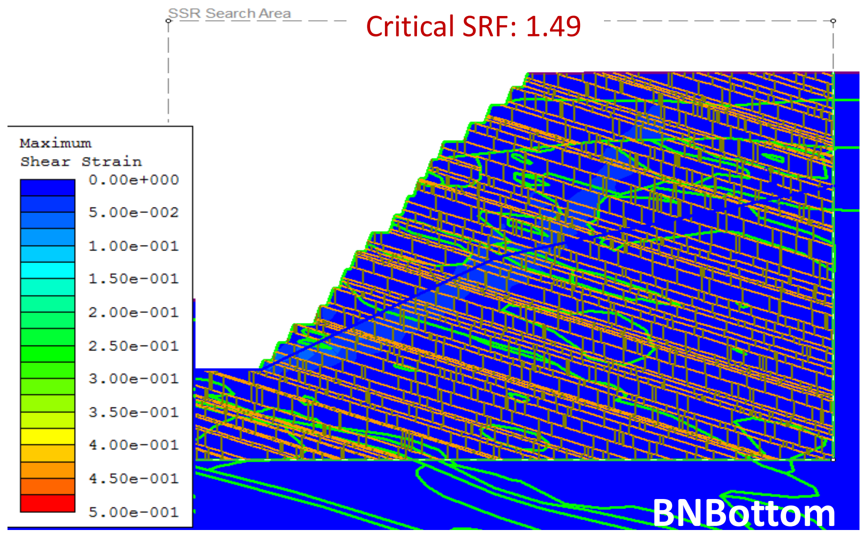

SSR Search Area
Critical SRF: 2.13
Maximum
Shear Strain
0.00e+000
4.00e-002
8.00e-002
1.20e-001
1.60e-001
2.00e-001
2.40e-001
2.80e-001
3.20e-001
3.60e-001
4.00e-001
BNUpper

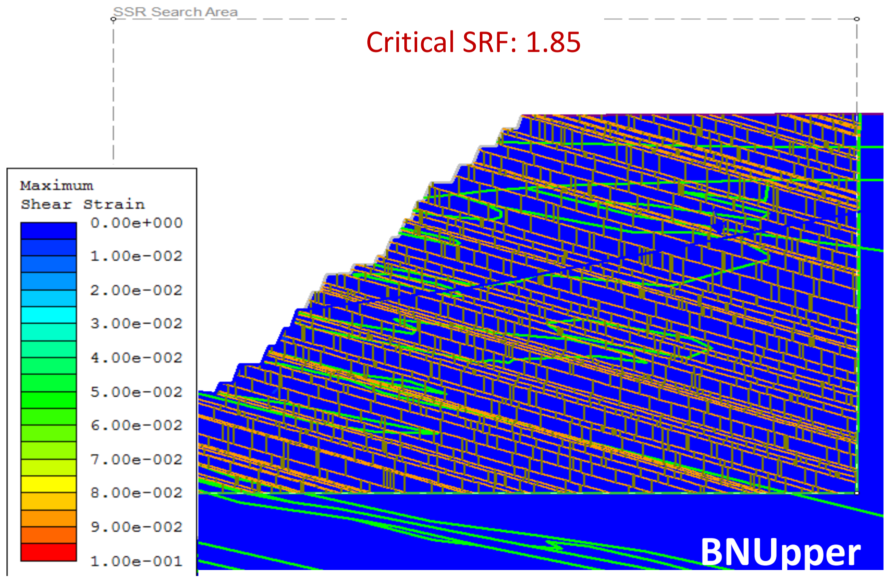

SSR Search Area
Critical SRF: 1.50
Maximum
Shear Strain
0.00e+000
6.00e-003
1.20e-002
1.80e-002
2.40e-002
3.00e-002
3.60e-002
4.20e-002
4.80e-002
5.40e-002
6.00e-002
Mideast

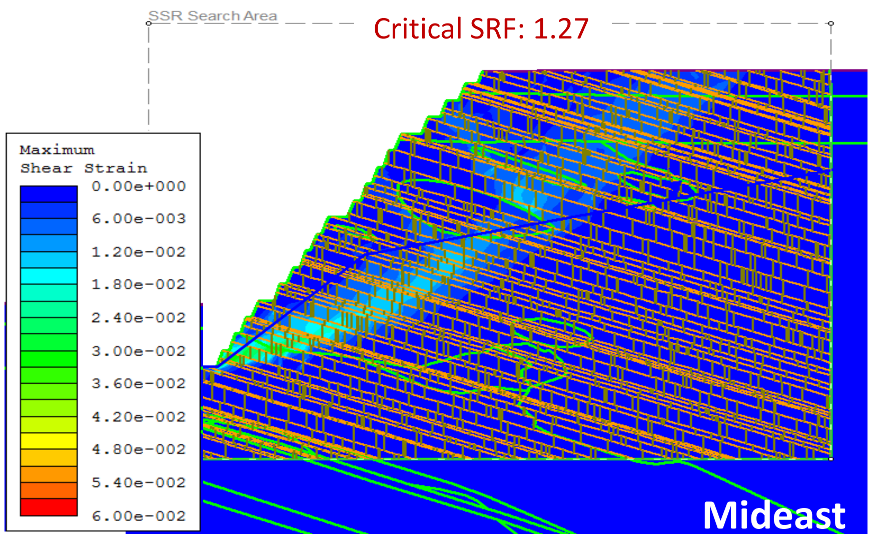

SSR Search Area
Critical SRF: 1.92
Maximum
Shear Strain
0.00e+000
1.80e-002
3.60e-002
5.40e-002
7.20e-002
9.00e-002
1.08e-001
1.26e-001
1.44e-001
1.62e-001
1.80e-001
North East

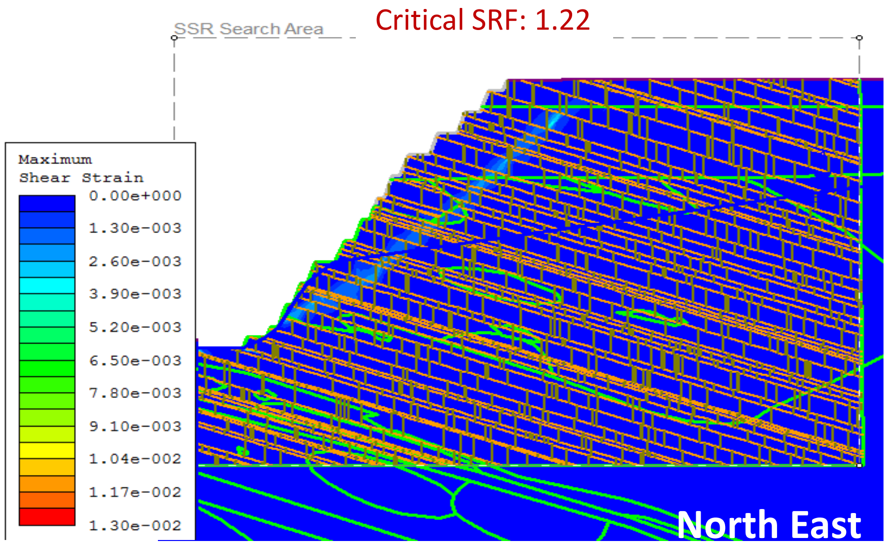


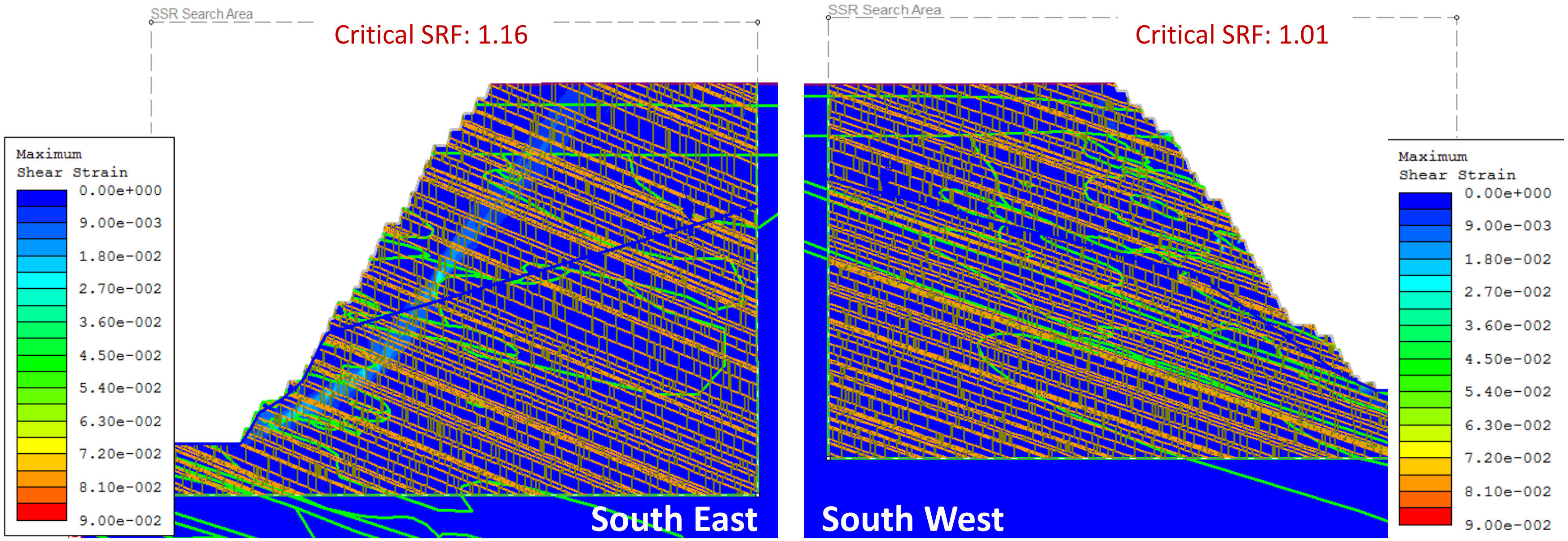
SSR Search Area
Critical SRF: 1.57
Maximum Shear Strain
0.00e+000
1.10e-002
2.20e-002
3.30e-002
4.40e-002
5.50e-002
6.60e-002
7.70e-002
8.80e-002
9.90e-002
1.10e-001
South East
SSR Search Area
Critical SRF: 1.08
South West
Maximum Shear Strain
0.00e+000
1.70e-002
3.40e-002
5.10e-002
6.80e-002
8.50e-002
1.02e-001
1.19e-001
1.36e-001
1.53e-001
1.70e-001


SSR Search Area
Critical SRF: 1.49
Maximum
Shear Strain
0.00e+000
5.00e-002
1.00e-001
1.50e-001
2.00e-001
2.50e-001
3.00e-001
3.50e-001
4.00e-001
4.50e-001
5.00e-001
BNBottom


SSR Search Area
Critical SRF: 1.85
Maximum
Shear Strain
0.00e+000
1.00e-002
2.00e-002
3.00e-002
4.00e-002
5.00e-002
6.00e-002
7.00e-002
8.00e-002
9.00e-002
1.00e-001
BNUpper

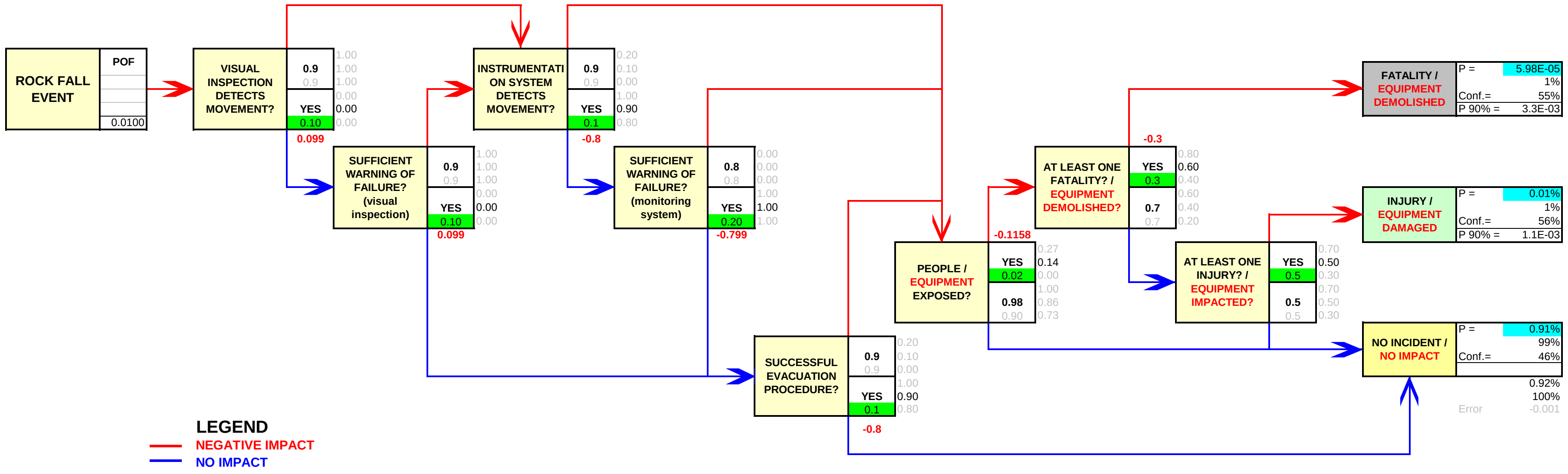

SSR Search Area
Critical SRF: 1.27
Maximum
Shear Strain
0.00e+000
6.00e-003
1.20e-002
1.80e-002
2.40e-002
3.00e-002
3.60e-002
4.20e-002
4.80e-002
5.40e-002
6.00e-002
Mideast


SSR Search Area
Critical SRF: 1.22
Maximum
Shear Strain
0.00e+000
1.30e-003
2.60e-003
3.90e-003
5.20e-003
6.50e-003
7.80e-003
9.10e-003
1.04e-002
1.17e-002
1.30e-002
North East


SSR Search Area
Critical SRF: 1.16
Maximum
Shear Strain
0.00e+000
9.00e-003
1.80e-002
2.70e-002
3.60e-002
4.50e-002
5.40e-002
6.30e-002
7.20e-002
8.10e-002
9.00e-002
South East
SSR Search Area
Critical SRF: 1.01
South West
Maximum
Shear Strain
0.00e+000
9.00e-003
1.80e-002
2.70e-002
3.60e-002
4.50e-002
5.40e-002
6.30e-002
7.20e-002
8.10e-002
9.00e-002

Appendix D: Risk Analysis

RISK ASSESSMENT OTJIKOTO
EVENT TREE FOR CONSEQUENCE ANALYSIS
SAFETY & EQUIPMENT



RISK ASSESSMENT OTJIKOTO
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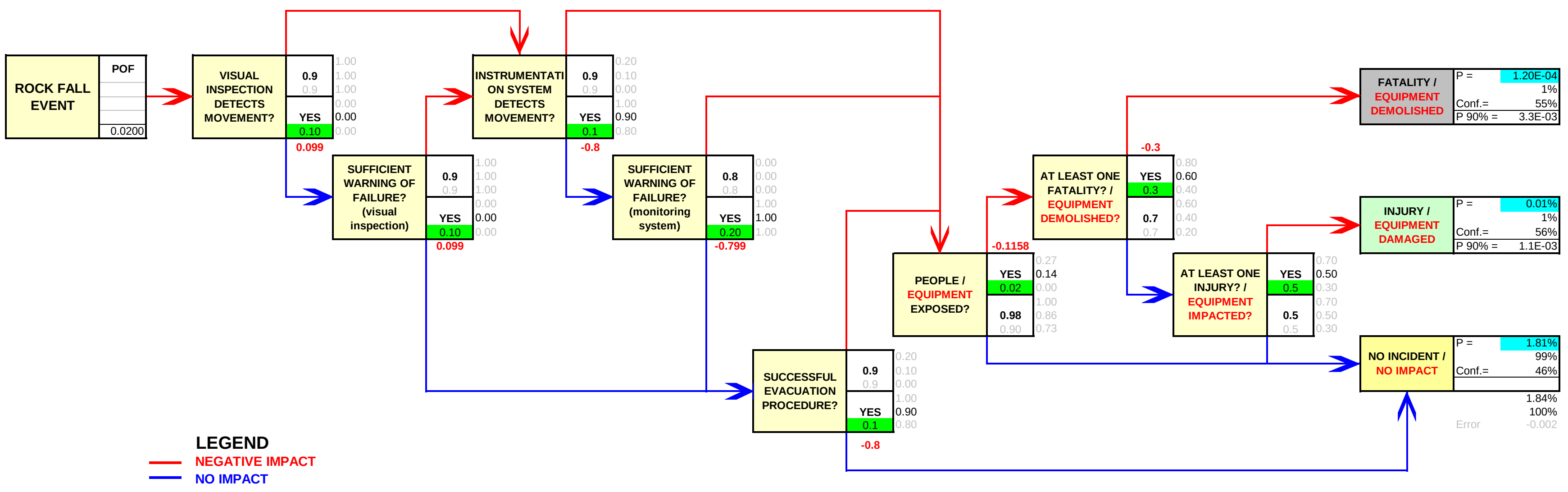
SAFETY & EQUIPMENT



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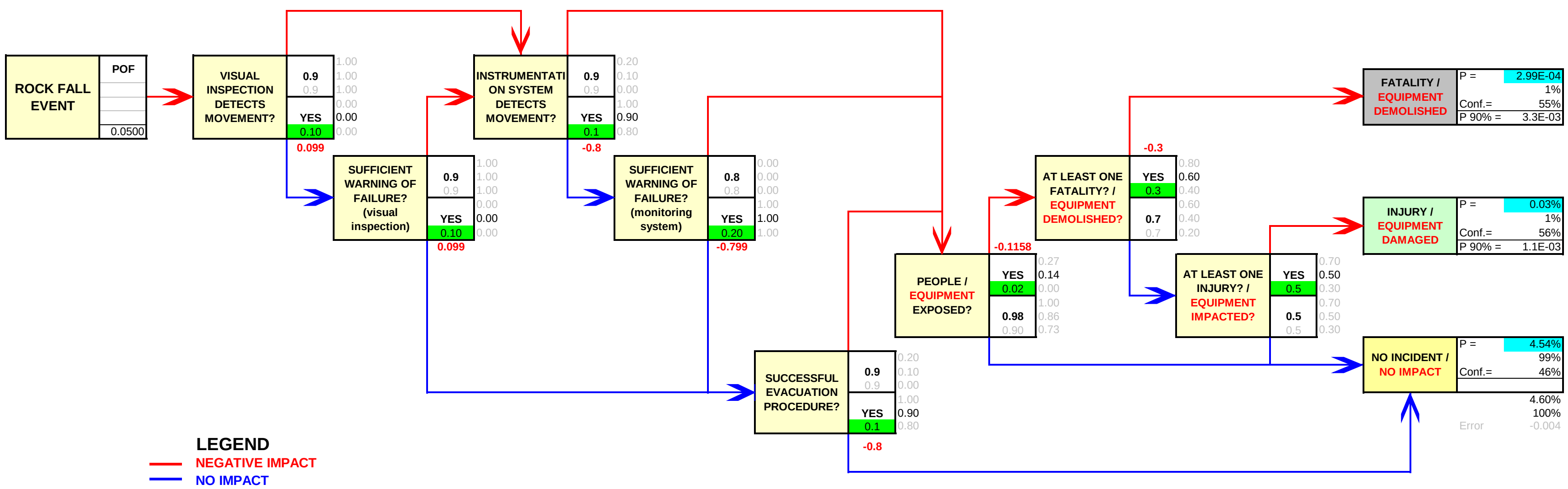

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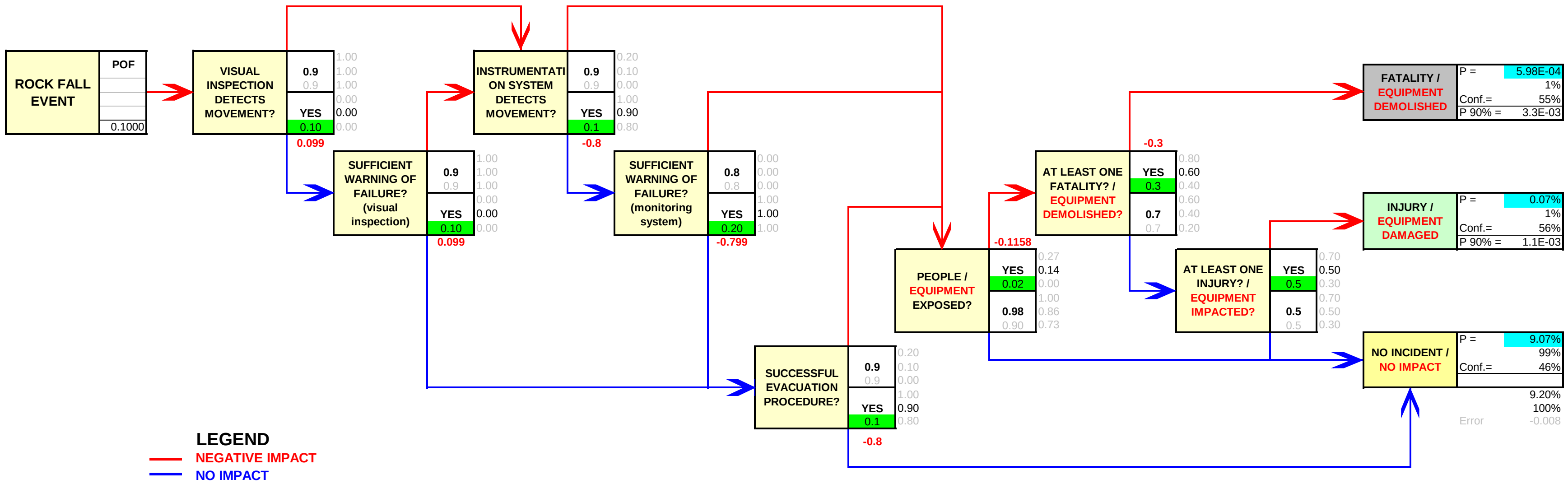

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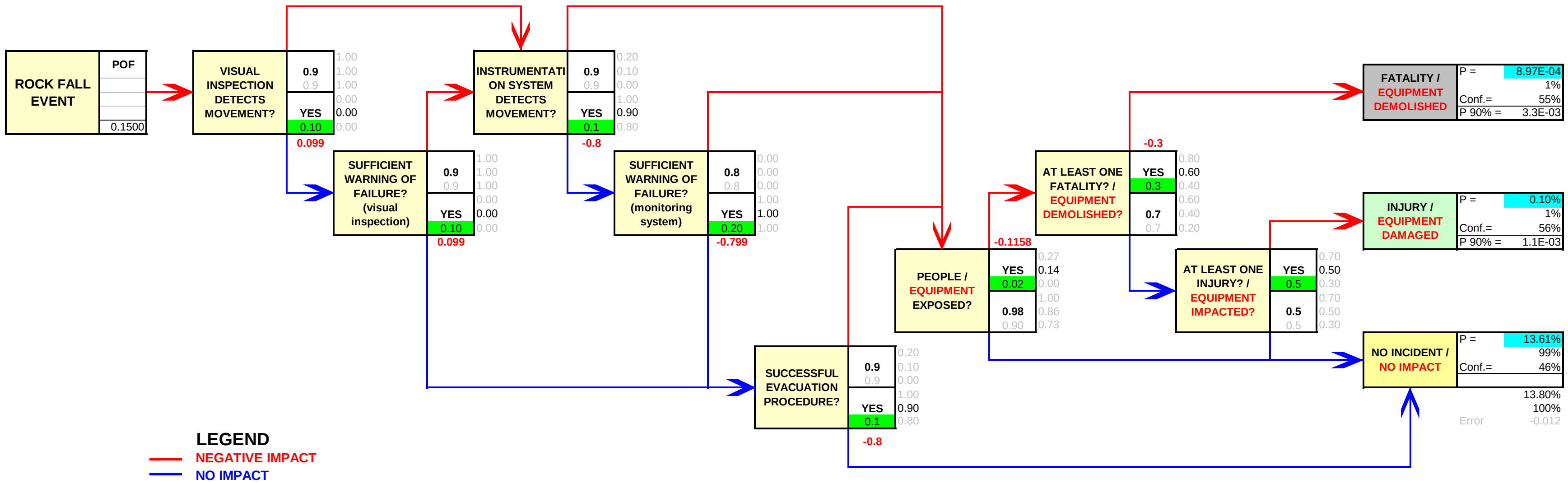

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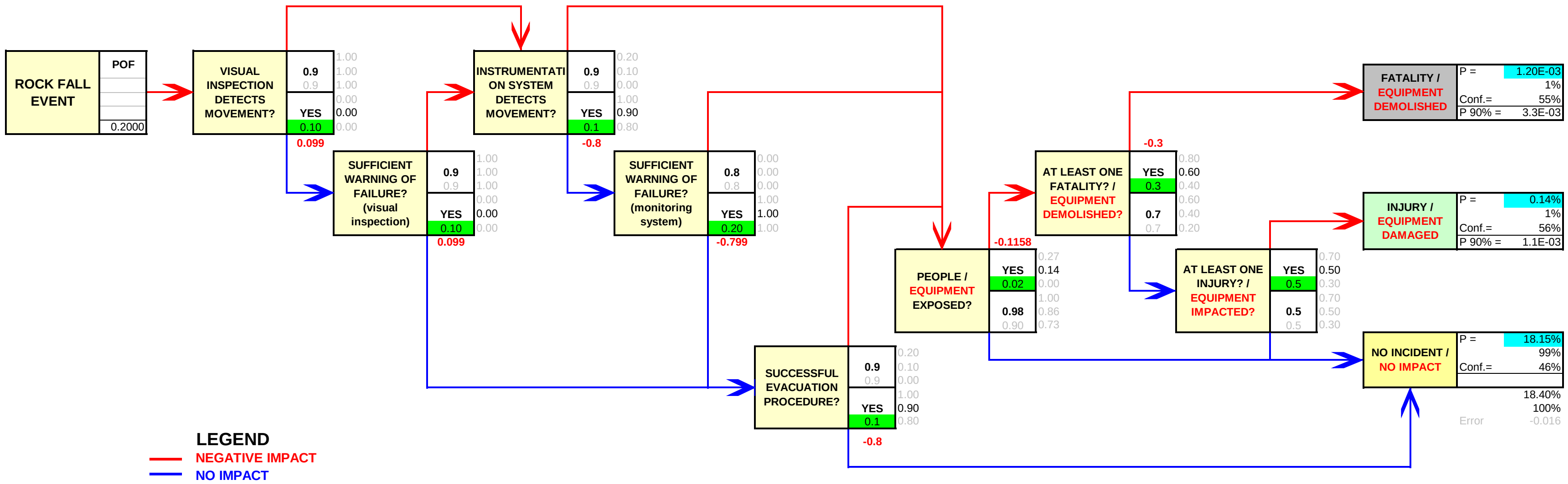

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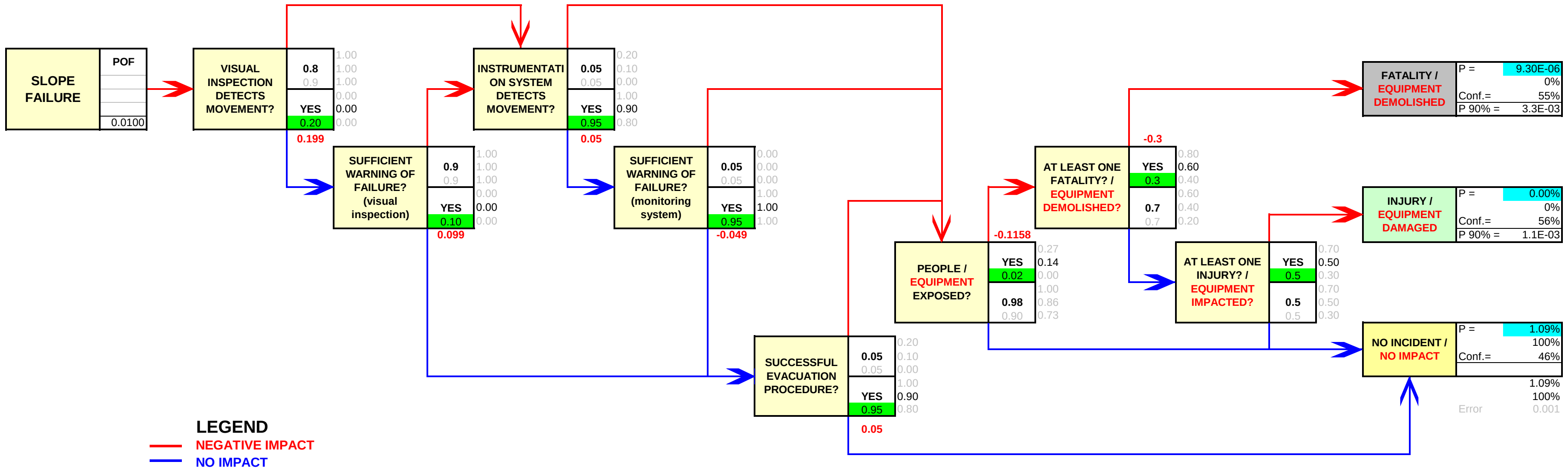

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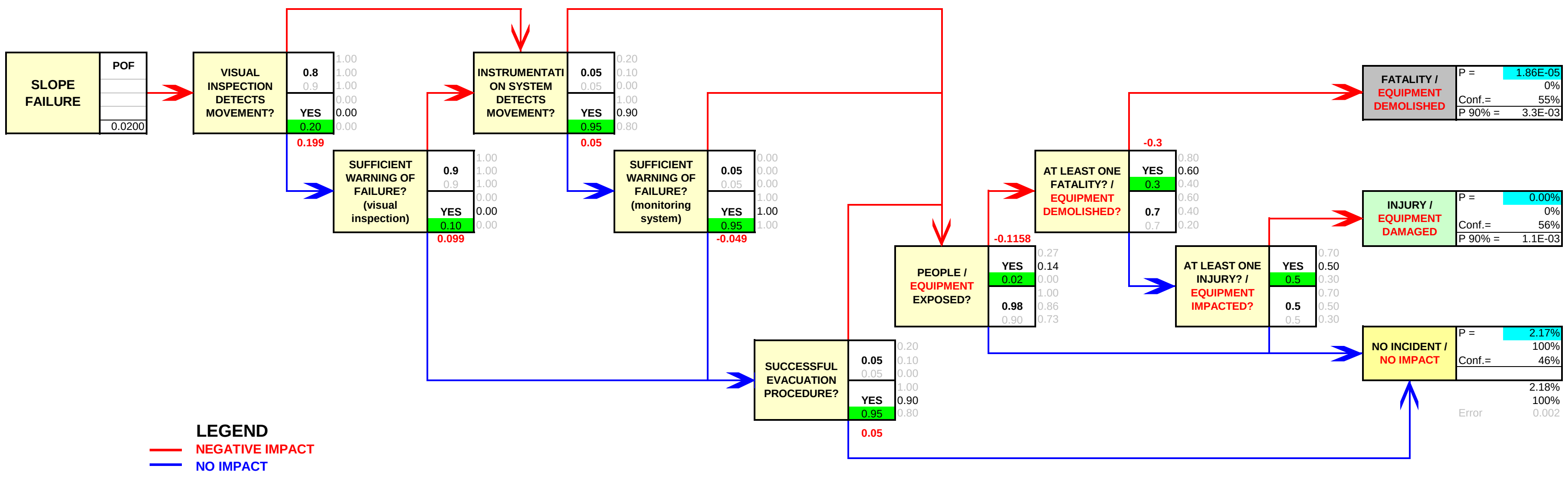

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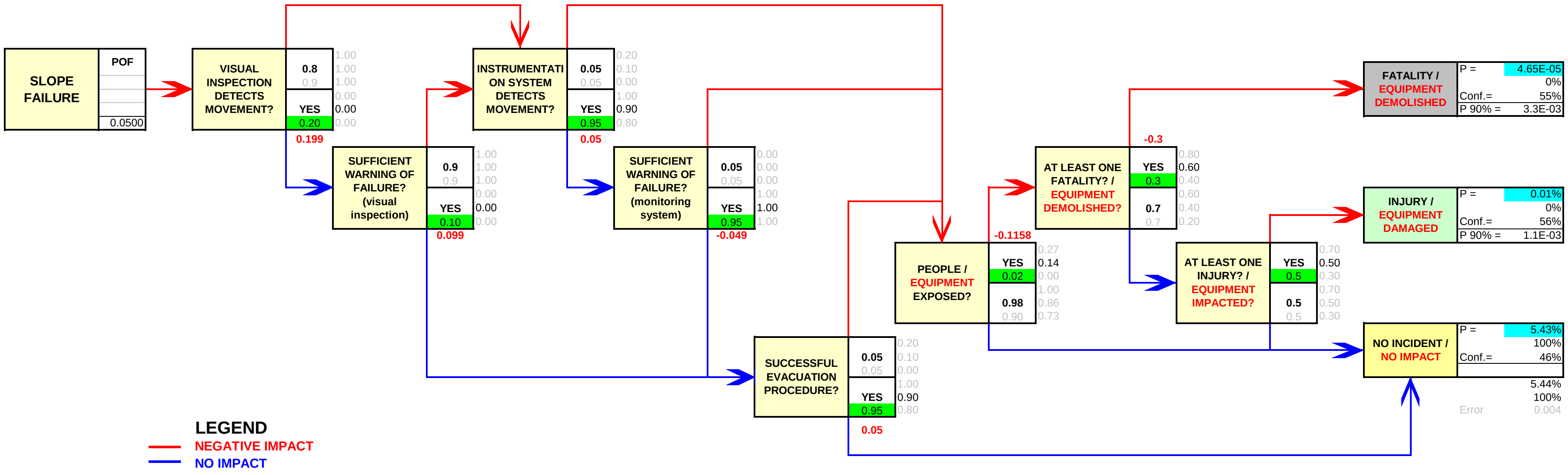

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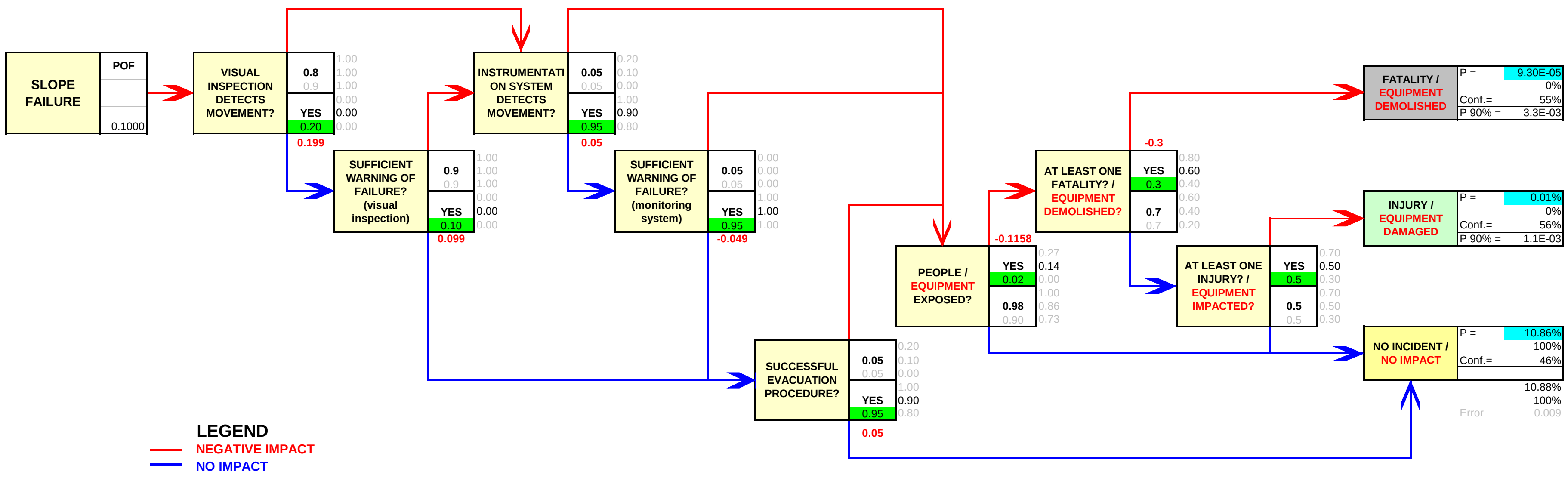

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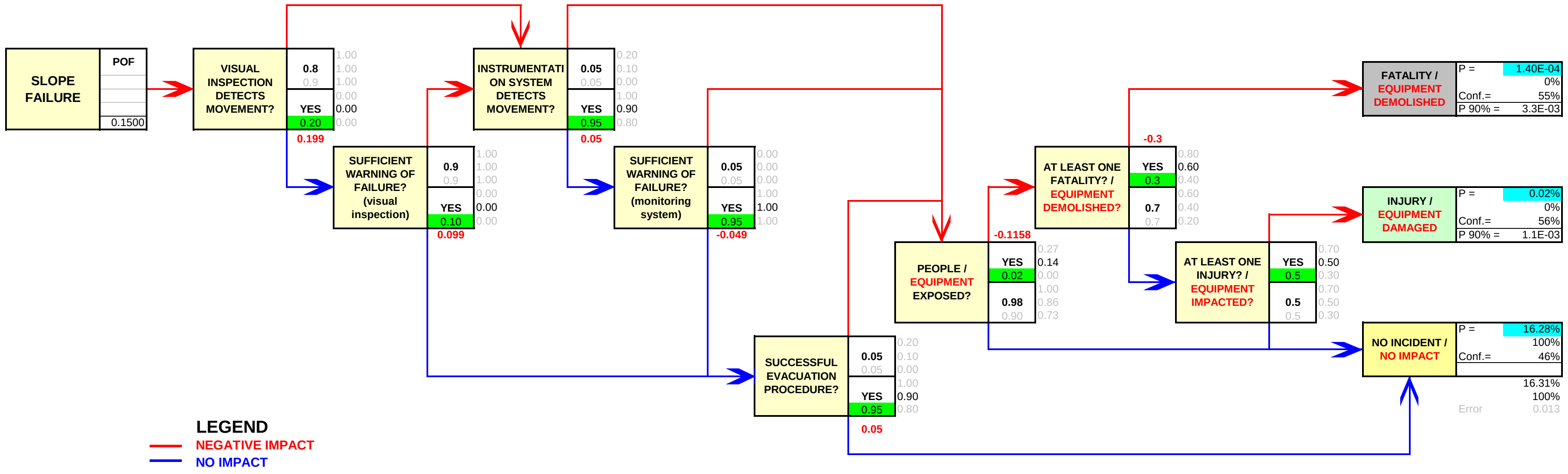

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