Division of Engineering and Science

Department of Civil Engineering

The Use of the Rock Mass Variability Index in Material Property Selection for Pit Slope Stability Modelling

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DECLARATION

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university.

To the best of my knowledge and belief this thesis contains no material previously published by any other person except where due acknowledgement has been made.

The following publications have resulted from the work carried out for this degree. Copies of selected published or in print refereed journal papers are presented in the appendix.

REFEREED JOURNAL AND CONFERENCE PAPERS

- *Narendranathan. and Nikraz . H; The Benefits of Probabilistic Slope Design in Contemporary Open Pit Mining, International Conferences on Advances in Geotechnical Engineering 2011.*
- *Narendranathan. and Nikraz . H; Optimal System Design for Instrumented Slope Monitoring in Open Pit Mines, International Conferences on Advances in Geotechnical Engineering 2011.*
- *Narendranathan. and Nikraz Optimal Modelling of Rock Slope Shear Strength Parameters within Foliated Rock Masses, International Conferences on Advances in Geotechnical Engineering 2011.*

ABSTRACT

The Use of the Rock Mass Variability Index in Material Property Selection for Pit Slope Stability Modelling

The primary aim of this research is to put forward a new process which would enhance the methodology which rock mechanics practitioners apply to formulate material parameters for the purpose of slope stability modelling. Often a great deal of emphasis is placed on the actual modelling process itself i.e. the type of software number of nodes etc but very little consideration is given to what is actually being fed into these software packages.

The Rock Mass Variability Index is a method being introduced by the author to aid practitioners in the process of calculating material properties for the purposes of stability modelling based on assessing and quantifying their statistical variability. To date though, there have been many different variations of probabilistic slope design approaches put forward by others; there is not one unified system to assess material strength variability. This is left to the discretion of the particular practitioner.

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i. Professor (Dr) Hamid Nikraz, Head of School of Curtin University of Technology Civil Engineering Department, for providing his assistance and guidance over the course of this research.

ii. Coffey Mining Pty Ltd, for providing the author access to their modelling software and packages.

iii. I would like to dedicate this work to my daughter, Tharani Isabella Narendranathan for being patient through this long process and having putting up with all the trials and tribulations associated with compiling a PhD dissertation.

1 INTRODUCTION

1.1 Back Ground to Research

The current economic climate and demand for resources has resulted in mine operators being prepared to accept aggressive slope designs and associated failures provided that they can be quantified (i.e. volume of failure, and clean up costs etc) and incorporated into the mining costs, at a feasibility study level.

This has numerous implications for pit slope design / geomechanics. Typically when geotechnical engineers conduct slope stability modeling a single value, usually the average or mean value for model input parameters (i.e. structural orientations, material properties and hydrological conditions) are selected.

The implications resulting from using a single value for model input parameters will vary depending on how the modeler has chosen the value, i.e. should the modeler choose a mean value from a log-normal distribution of material strengths an optimistic representation of the stability may eventuate, consequently if the designer decides to conservatively downgrade the values by a nominal factor, a pessimistic representation of slope angles may eventuate, which would result in excessive stripping and mine development costs.

A more realistic and representative way to model the inherent variability of geological / geotechnical conditions would be to do so probabilistically. Whereby the modeler can assess the variability of the geological structure as observed / recorded during the core logging process, and a similar approach can be applied for the selection of material strength properties.

The final model output would be a distribution of factors of safety in relation to overall slope angles, i.e. probabilities of failure for the respective slope geometries could be determined. This output chart could be modified by the mining engineer to incorporate mining costs as well as clean up costs for the respective slope

geometries in relation to their respective anticipated failure volumes i.e. probabilities of failure.

This research outlines the detailed processes involved in deriving a geotechnical (pit slope) design, i.e. starting from the data collection phase (core logging / face mapping), through to the data interpretation and formulation of slope parameters, and determining the variability and uncertainties associated with each of the process and ensuring that they are adequately captured within the model input parameters, and how the mining engineer could utilise this information to derive final feasibility study mining costs.

1.2 Objectives of Research

This research thesis will provide a contribution to the field of Civil Engineering / Rock Mechanics with particular regard to the selection of material parameters when modelling the stability of rock and soil slope within open pit mines. The objectives for this research study can be outlined as follows:

1. Outlining the fundamentals of rock and soil slope design; in particular a quantitative review of the importance and criticality of material property selection.
2. Description of commercially available slope stability modeling packages.
3. Description of the various theories of empirically evaluating rock mass and rock mass defect plane strengths, based on information obtained from field methods i.e. core logging and exposure (surface, wall) mapping.
4. Description of testing procedures for analytically assessing rock mass and rock mass defect plane strength properties.
5. Calculating the statistical significance and confidence of the respective data sets, i.e. the empirical and analytical data sets respectively will be quantified using a methodology devised by the author know as the Rock Mass Variability Index.

6. Four case studies will be presented in the subsequent comportment:

   - **Case Study A** – Will present data assessed from a moderately strong bedded sedimentary rock, with weak infilling material inter-bedded within the foliation planes. The author will present data gathered from empirical and analytical means which have been statistically quantified using the Rock Mass Variability Index.
   - **Case Study B** – Will detail the data assessed from a very weak highly weathered sedimentary shale rock. The author will present data gathered from empirical and analytical means which have been statistically quantified using the Rock Mass Variability Index.
   - **Case Study C -** Will present data assessed from a moderately strong foliated metamorphic rock, with high friction infilling material inter-bedded within the foliation planes. The author will present data gathered from empirical and analytical means which have been statistically quantified using the Rock Mass Variability Index.
   - **Case Study D -** Will present data assessed from a moderately strong foliated metamorphic rock, with weak clay (low friction) material inter-bedded within the foliation planes. The author will present data gathered from empirical and analytical means which have been statistically quantified using the Rock Mass Variability Index.

7. A comparison analyses of the statistical integrity of the respective data sources (empirical and analytical) will be undertaken for each of the rock types. The primary aim of the comparison analyses would be to determine the statistical confidence of each data source and assess a weighting coefficient using the Rock Mass Variability Index to determine its suitability for use as representative design parameters for stability modeling.

8. Undertake Limit Equilibrium modeling for the rock types identified within the four case studies, based on material properties evaluated using the Rock Mass Variability Index method, to calculate stable slope geometry for mining purposes.
9. Undertake sensitivity analyses on the rock types identified as part of the case studies, based on material properties evaluated using the traditional approach, i.e. solely using data from laboratory testing (analytical means), in comparison to the Rock Mass Variability Index methodology. As part of this the author will present the potential positive implications i.e. in terms of increased mine revenue due to optimised slope angles and reduced risk of slope failure that arises from using the Rock Mass Variability Index as opposed to traditional methods.
10. A detailed list of further work to complement that undertaken as part of this research will be presented.

1.3 Significance

The primary aim of this research is to put forward a new process which would revolutionalise the methodology which rock mechanics practitioners apply to formulate material parameters for the purpose of slope stability modeling. Often a lot of emphasis is put on the actual modeling process itself i.e. the type of software number of nodes etc but very little consideration is given to what is actually being fed into these software packages.

The Rock Mass Variability Index is a method being introduced by the author to aid practitioners in the process of calculating material properties for the purposes of stability modeling based on assessing and quantifying their statistical variability. To date though there have been many different variations of probabilistic slope design approaches put forward by others; there is not one unified system to assess material strength variability, this is left to the discretion of the particular practitioner.

This was the case a number of years ago with empirical rock mass assessments, whereby there was not one unified method of describing rock mass conditions based on the observations made from drill core or face mapping. It was not till 1976 that Bieniawski proposed his Rock Mass Rating (RMR_{76}) system to describe rock masses which shifted the focus of the rock mechanics industry to a quantitative approach rather than a qualitative approach to describing rock conditions, this approach was readily accepted as it provided an unified method to compare different rock types and effectively quantify the difference within numerical modeling packages.

It is the author's intention that the Rock Mass Variability will in future be accepted by rock mechanics industry practitioners as a method for quantifying the statistical uncertainty and the level of confidence of a particular data set subsequent to undertaking stability modeling.

1.4 Identification of Research Needs

It is the author's contention that where laboratory results are used to determine material strength parameters for strong rock i.e. Unconfined Compressive Strengths greater than 50Mpa there is the possibility of underestimating material strengths, as typically laboratory testing facilities are limited in this regard.

Whereby an Unconfined Compressive Strength test is limited by the strength of the steel within the test frame, it is feasible to have rock strengths greater than the strength of steel i.e. as high as 450Mpa.

The shear strength of rock material is assessed by undertaking triaxial tests, however typically confinement pressures greater than 30Mpa cannot be applied in the test cells, hence materials stronger than 100Mpa cannot be effectively evaluated. Note that typically confinements stress of up to one third the Unconfined Compressive Strength should be applied when undertaking a triaxial test.

Where weak materials are being evaluated i.e. materials with Unconfined Compressive Strengths in the order of 1Mpa to 15Mpa. The use of laboratory results will tend to overestimate the strength of the materials; this is due to the fact the majority of the extremely weak samples would have typically disintegrate during manual handling and transportation.

Thus the samples actually tested for the purposes of strength evaluation provide an upper bound estimate of strength; this invariably results in an optimistic slope design which may not be implementable. The implications of this for an operating mine would be as follows:

- **Clean up costs** – The cost of mobilising and operating equipment, which would otherwise be used for mine production to clean up failure masses on the pit floor, so that regular mining activities can resume.
- **Slope re-formation** – Depending on the type of failure mechanisms, for example rotational failure, steep back scarps may be left behind, that require battering back (flattening) so as not to initiate further failures, this is sometimes carried out in parallel with regular pit floor cleanup activities.
- **Haul road repair and re-access** – Should a particular slope failure, take place onto a haul ramp or compromise its integrity by occurring below it and undercutting it, access to the pit will be lost until a alternate access can be restabilised, if the ramp is the sole access to the pit. This would result in significant production losses albeit temporarily.
- **Unrecoverable ore** – In an extreme case there is the potential for a slope failure to bury an orebody or at least a critical blend required for processing purposes, and the consequences of this failure becomes significant as production ceases entirely until the area is remediated and the ore recovered.

- **Damage to equipment, personnel and infrastructure** – Where key pieces of infrastructure (decline portals, process facilities, public roads and highways etc), equipment or personnel are damaged or injured the consequences can be extreme.

1.5 Research Method

This research aims to investigate the use of probabilistic slope design and how it can be applied to designing pit slopes within a particular rock mass. The author intends to cover a broad range of slope analyses methods and their applicability to various slope geometries and design scenarios.

The author will present case studies illustrating the requirement for a standardised method of selecting material properties for slope stability modeling. These case studies will quantify the cost savings to the overall slope and mine development as well as the potential hazards and increased risks of instability due to the incorrect or non representative selection of material parameters.

These case studies will quantitatively and compare the outcomes as a result of applying the conventional methods of material parameters selection, which basically utilises laboratory testing information and does not consider or pays very little note to data interpreted using empirical methodologies, with the Rock Mass Variability Index methodology.

The following aspects of probabilistic slope design methodology will be covered in detail as part of the case studies:

- Geotechnical field data collection;
- Laboratory testing;
- Interpretation and analyses of data for slope designs; and
- How these results may be utilised within mine optimisation software packages like Whittle when conducting economic analyses.

One of the novel items being presented is the use of the Rock Mass Variability Index. This is a statistical methodology being proposed by the author for the selection of material properties when undertaking stability modeling. To date in the rock mechanics field there has not been a standardised approach for the selection of material parameters.

Usually this process is unique to the particular practitioner hence the outcome of the analyses can vary significantly at times. It is the author's intention to put forward a methodology and process which will be accepted by the rock mechanics community as a standardised approach for the selection of material properties as input parameters for slope stability modeling.

It is the author's intention to demonstrate that the use of the proposed Rock Mass Variability Index will alleviate (as far as practicable) uncertainty in the calculated slope design and will provide the most 'fit for purpose' parameters which can be implemented with minimal risk of instability or failure, and where there exists some uncertainty it can be accounted for within the model.

One of the major aims is to present a scenario based analysis, through the use of case studies to draw the reader's attention to the pit falls of traditional approaches in selecting material properties based solely on laboratory results.

1.6 Thesis Organisation

The thesis has been divided into chapters to highlight the various aspects of the study:

Chapter 1 consists of an introduction into the entire research study in general and highlights the needs, scope, objectives, methodology and organisation of the study.

Chapter 2 consists of literature of the following: slope design methodology, slope rock mass assessment techniques, slope stability modeling techniques and slope geometry calculation methods.

Chapter 3 presents the introduction to the four case studies being considered as part of this research. The concept of calculating the Rock Mass Variability Index, as proposed by the author is introduced to the reader.

Chapter 4 presents the data gathered from empirical and analytical means as Case Study A.

Chapter 5 presents the data gathered from empirical and analytical means as Case Study B.

Chapter 6 presents the data gathered from empirical and analytical means as Case Study C.

Chapter 7 presents the data gathered from empirical and analytical means as Case Study D.

Chapter 8 presents the methodology by which rock mass and defect plane parameters were assessed for the four lithologies.

Chapter 9 presented the application of the Rock Mass Variability Index to the results obtained from Chapter 8 to choose design material strengths for modeling.

Chapter 10 presents the results of Limit Equilibrium modeling and calculated slope geometry from case studies A, B, C and D based on the Rock Mass Variability Index methodology.

Chapter 9 presents the results of a sensitivity analyses and cost benefit analyses for case studies A, B, C and D, comparing the slope geometry as derived using the Rock Mass Variability Index methodology with that obtained using traditional approaches.

Chapter 10 concludes this study and details further work to be undertaken to complement and further verify the foundation on which the concept of the Rock Mass Variability Index is based.

2 LITERATURE REVIEW

Probabilistic design is a discipline within engineering design which is becoming more readily applied in industry in recent years due to the advent of modern computers. It deals primarily with the consideration of the effects of random variability upon the performance of an engineering system during the design phase. Typically, these effects are related to quality and reliability of the data.

Thus, probabilistic design is a tool that is mostly used in areas that are concerned with quality and reliability. For example, product design, quality control, systems engineering, machine design, civil engineering (particularly useful in limit state design) and manufacturing. It differs from the classical approach to design by assuming a small probability of failure instead of using the safety factor.

When using a probabilistic approach to design, the designer no longer thinks of each variable as a single value or number. Instead, each variable is viewed as a probability distribution.

From this perspective, probabilistic design predicts the flow of variability (or distributions) through a system. By considering this flow, a designer can make adjustments to reduce the flow of random variability, and improve quality. Proponents of the approach contend that many quality problems can be predicted and rectified during the early design stages and at a much reduced cost.

Typically, the goal of probabilistic design is to identify the design that will exhibit the smallest effects of random variability, as will be later presented within the case studies i.e. the expected case design scenario for the respective slope sectors.

This could be the one design option out of several that is found to be most robust. Alternatively, it could be the only design option available, but with the optimum combination of input variables and parameters.

A detailed overview is presented of the processes involved in deriving a geotechnical design, i.e. starting from the data collection phase (core logging / face mapping), through to the data interpretation and formulation of slope parameters, and determining the variability and uncertainties associated with each of the process and ensuring that it is adequately captured in the model input parameters, and how the mining engineer could utilise this information to derive final feasibility study mining costs.

A proper economic analysis of pit slope design must reflect the trade-off between the benefits and the increased risk of slope failure inherent in steeper slope angles. This research describes the findings of case studies in which a probabilistic economic analysis procedure was applied to the selection of pit slope design geometries for open pit mines.

It is anticipated that this research would provide an adequate method for the quantification of uncertainty associated with rock masses hence enabling the slope engineer to provide a slope design where its reliability is known (quantified).

There are a number of benefits that arise as a result of applying this design process particularly for mines with high stripping ratios (i.e. narrow vein gold mines) as in recent times due to the falling metal prices operating costs have to be kept to a minimum.

These economic benefits are summarised below:

- Once the variability of the design has been assessed, expected volumes of failure can be calculated. This information can be fed into 'fed' into a pit optimisation programme e.g. Whittle to provide a pit shell(s) based on the optimum slope angle given a set of parameters, be they with the geotechnical, geological or financial.

- This would facilitate better scheduling requirements, i.e. as the mine scheduling engineer now has an indication of the expected volumes of failure so he can make an estimate of clean up times and equipment requirements.
- The uncertainty associated with the geotechnical model can be inputted into spreadsheets that calculate Net Present Value (NPV) and Internal Rate of Return (IRR) or any other method for tracking performance, which would provide mine management a realistic process to track and forecast operating costs and performance.

2.1 Introduction to Rock Slope Design Methodology

In designing the very large excavated slopes which are becoming increasingly common in both mining and civil engineering projects, the engineer is faced with two conflicting requirements, as follows, after Hoek and Bray (1973):

1. The vast sums of money that can be saved by steepening the slopes, thereby reducing the amount of material to be excavated; and
2. The loss of life and serious damage to property that can result from excessive steepening of a particular slope.

As the rock mass behind each slope is unique, there are no standard recipes or routine solutions which are guaranteed to produce the right answer each time they are applied. A practical solution is built up from the basic geological data, rock strength information, groundwater observations and engineering judgement. Hoek and Bray, (1973).

It is the author's intention to present a number of these tools and techniques and to illustrate their application demonstrated by the undertaking of case studies which will be presented in the subsequent sections. This section presents the historical

evolution of some of these techniques and how they might be applied in present day slope stability assessments.

2.2 Historical Evaluation of Rock Mechanics

We tend to think of rock engineering as a modern discipline and yet, as early as 1773, Coulomb included results of tests on rocks from Bordeaux in a paper read before the French Academy in Paris, Heyman (1972).

French engineers started construction of the Panama Canal in 1884 and this task was taken over by the US Army Corps of Engineers in 1908. In the half century between 1910 and 1964, 60 slides were recorded in cuts along the canal and, although these slides were not analysed in rock mechanics terms, recent work by the US Corps of Engineers (Lutton et al, 1979) shows that these slides were predominantly controlled by structural discontinuities and that modern rock mechanics concepts are fully applicable to the analysis of these failures. The effect of geological structure and its implication to slope stability and geometry will be discussed in greater detailed within Section 2.3.

In discussing the Panama Canal slides in his presidential address to the first international conference on Soil Mechanics and Foundation Engineering in 1936, Karl Terzaghi, Terzaghi, (1936), and Terzaghi and Voight, (1979) said ‘The catastrophic descent of the slopes of the deepest cut of the Panama Canal issued a warning that we were overstepping the limits of our ability to predict the consequences of our actions’.

In 1920 Josef Stini started teaching ‘Technical Geology’ at the Vienna Technical University and before he died in 1958 he had published 333 papers and books, Mülle, (1979). He founded the journal Geologie und Bauwesen, the forerunner of today’s journal Rock Mechanics, and was probably the first to emphasise the importance of structural discontinuities on the engineering behaviour of rock masses.

Other notable scientists and engineers from a variety of disciplines did some interesting work on rock behaviour during the early part of this century. Von Karman (1911), King (1912), Griggs (1936), Ide (1936), and Terzaghi (1945) all worked on the failure of rock materials. In 1921 Griffith proposed his theory of brittle material failure and, in 1931 Bucky started using a centrifuge to study the failure of mine models under simulated gravity loading.

None of these persons would have classified themselves as rock engineers or rock mechanics engineers, the title had not been invented at that time but all of them made significant contributions to the fundamental basis of the subject as we know it today.

The author would like to point out that the above list is merely a small sample of the research work done in the field of rock mechanics prior to 1950. The reader should appreciate that there are a number of other authors who have contributed significantly to the field of rock mechanics during this time.

The early 1960s were very important in the general development of rock engineering world-wide because a number of catastrophic failures occurred which clearly demonstrated that, in rock as well as in soil, 'we were over-stepping the limits of our ability to predict the consequences of our actions' Terzaghi and Voight, (1979).

Probabilistic slope design has been in practise for some time (at least 20 years or so), having been pioneered by Lilly, Martin, Piteau, McMahon and others. However in recent years published material tends to deal with singular components of overall probabilistic slope design for example specifically focusing on the analysis of kinematic failure mechanisms, without necessarily providing an overall appreciation of how it fits into the 'larger picture' i.e. the overall stability of a mine slope geometry.

The author intends this thesis to provide the reader with a comprehensive understanding of deterministic slope design and an appreciation of how probabilistic

slope design can be carried out and its potential benefits within the overall mine design and optimisation process.

This research provides an overview of the probabilistic slope design methodology, as can be applied to the undertaking of mine geotechnical feasibility studies (discussed in greater detail within Section 3.0). Specifically the following will be addressed:

- Geotechnical field data collection;
- Laboratory testing;
- Geotechnical model formulation;
- Interpretation and analyses of data for slope designs; and
- How these results may be utilised within mine optimisation software packages like Whittle when conducting economic analyses.

One of the most challenging aspects of conducting a slope stability analysis is the uncertainty associated with the geotechnical model typically this level of uncertainty is catered for by incorporating a higher factor of safety within the slope design, which experienced slope design engineers have a 'feel' for. However this may prove somewhat challenging when dealing with rock masses where there is very little information present.

This may result in the engineer either allowing for 'too much' or 'not enough' thereby producing an overly conservative or overly optimistic slope design which may not be achievable.

It is the intention of this research to provide an adequate method for the quantification of this uncertainty hence enabling the slope design engineer to provide a slope design whose reliability or confidence level is known.

There are a number of benefits that arise as a result of applying this design process, particularly in recent times where costs have to be kept to a minimum, due to the

falling commodity prices. The case studies quantify the potential economic benefits that arise due to the application of this process.

2.3 Undertaking a Slope Stability Investigation for a Mine Feasibility Study

Undertaking a slope stability investigation can be an arduous task. There are many items to consider. Typically most investigation programmes would involve a data collection phase involving investigative drilling and laboratory testing, a data processing phase involving the collation of the data from the previous phase to formulate a geotechnical model, and finally the derivation of slope geometry based on the geotechnical model and stability analysis.

The author's recommends the following approach as put forward by Hoek et al (1973) when undertaking slope stability investigations is outlined in the flow chart below (Figure 2.3.1).

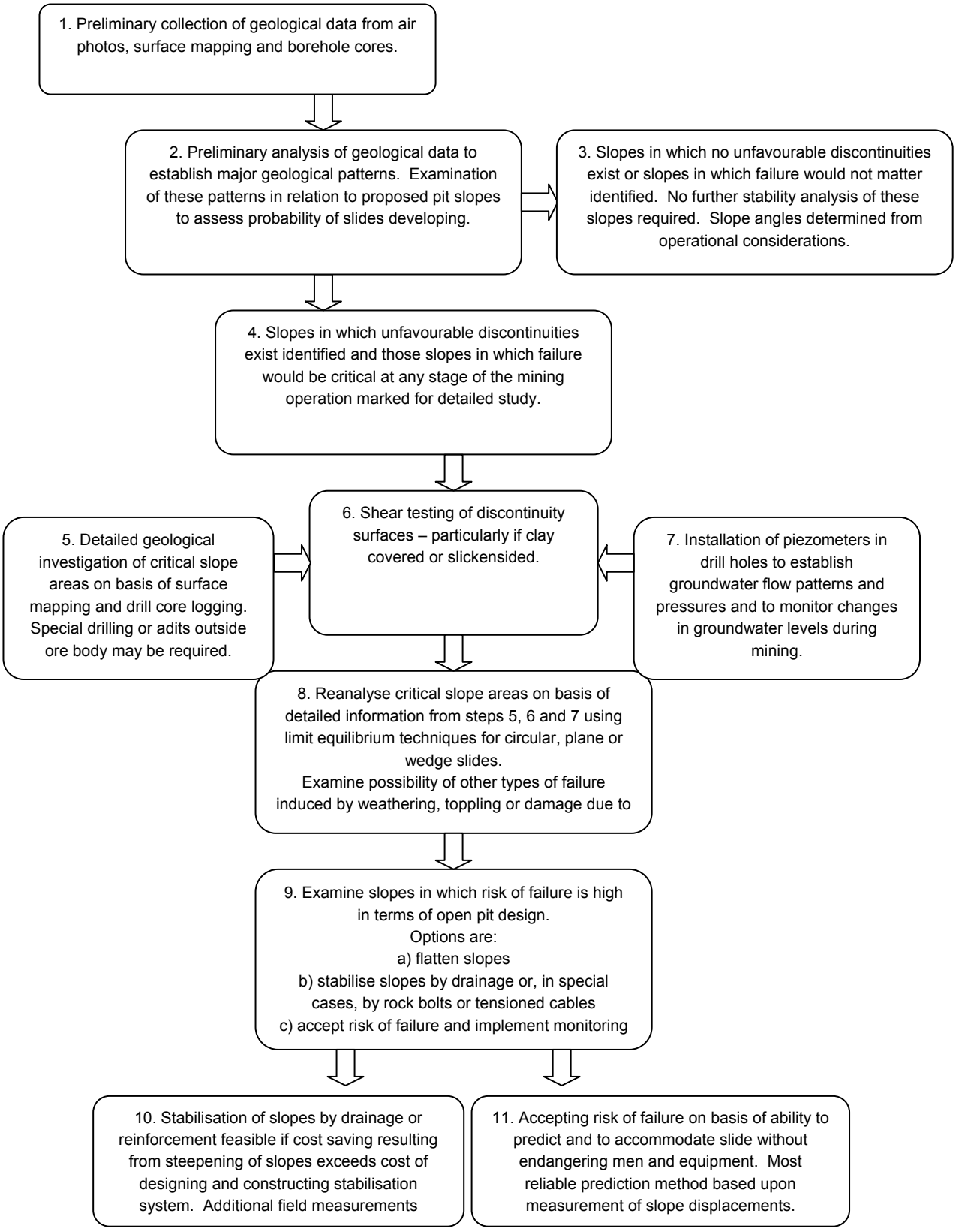


Figure 2.3.1 – Flow Chart for Undertaking Slope Stability Investigations

The preliminary investigations listed above under stage 1 should ideally be integrated into the evaluation and feasibility studies of the mine. Much of the information required for these preliminary slope studies can be obtained at minimal cost if provision is made for its collection during the exploration drilling phase of the project.

In existing open pit mines (brown-field projects), a much more detailed knowledge of the geology and of the areas of the pit which have shown signs of instability will already be available as opposed to a new site (green-field project).

Once critical slopes in a mine have been identified, the more detailed studies (analyses) which are necessary can be implemented, however these will vary so much from mine to mine hence general guide lines are very difficult to define.

This does not mean that the techniques or steps presented in Figure 2.3.1 for slope analysis are particularly difficult but it does mean that these analyses only provides part of the answer to a slope problem. It is up to the engineer to use judgement for refining these parameters during the operational phase of the project.

2.4 Discontinuous Rock Masses and its Implication on Slope Geometry

Stini was one of the pioneers of rock mechanics in Europe and he emphasised the importance of structural discontinuities in controlling the behaviour of rock masses, Müller (1979).

Stini was involved in a wide range of near-surface civil engineering works and it is not surprising that his emphasis was on the role of discontinuities since this was obviously the dominant problem in all his work.

Similarly, the text book by Talobre (1957), reflecting the French approach to rock mechanics, recognised the role of structure to a much greater extent than did the texts of Jaeger and Cook, Coates and Obert and Duvall.

A major impetus was given to this work by the Malpasset dam failure and the Vajont disaster mentioned earlier. The outstanding work by Londe and his co-workers in

France, Londe (1965), Londe et al, (1969, 1970) and by Wittke (1965) and John (1968) in Germany laid the foundation for the three-dimensional structural analyses which we have available today. Figure 2.4.1 shows a wedge failure controlled by two intersecting structural features in the bench of an open pit mine.

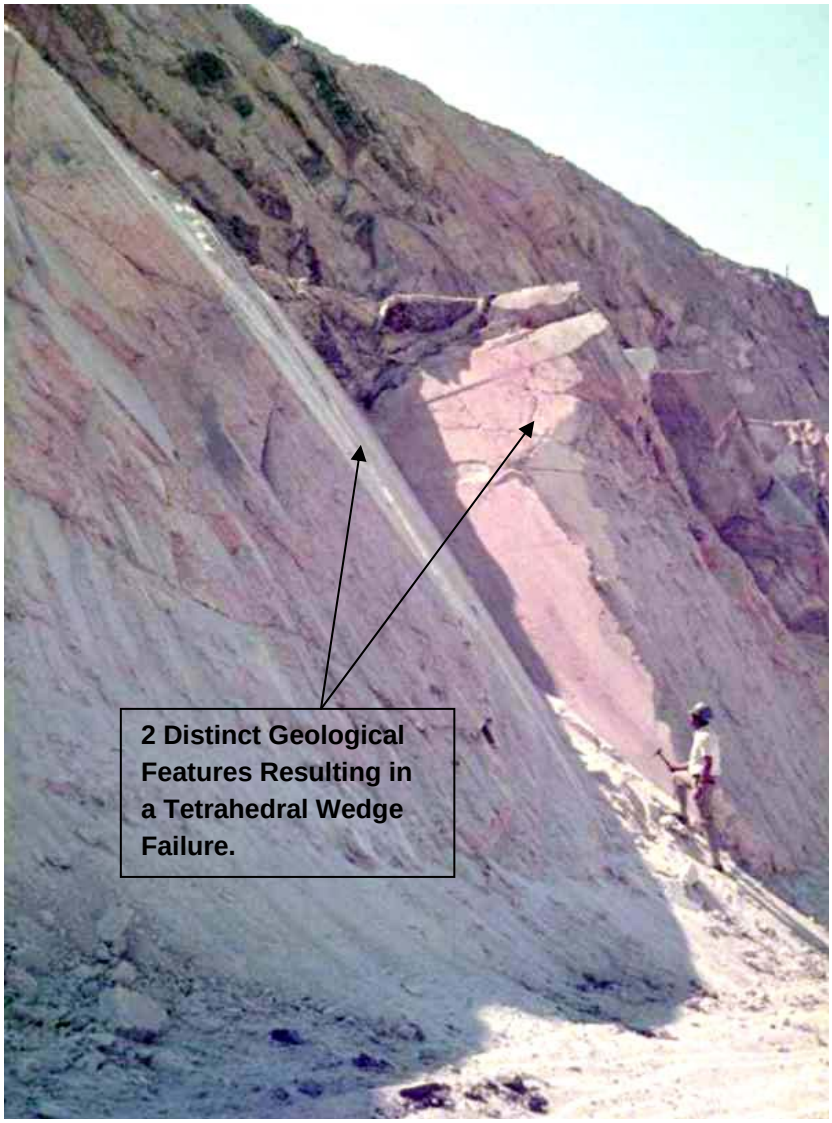


Figure 2.4.1: Wedge failure controlled by intersecting structural features

The importance of geological structure and its influence on slope geometry will be put into context in subsequent sections within this thesis, which discuss the importance of the various aspects of these features i.e. persistence and probability of occurrence etc within a pit slope.

2.5 Geotechnical Data Collection for a Probabilistic Stability Model

The formulation of a probabilistic geotechnical model starts at the data collection phase of a project, typically when collecting geotechnical information the following are essential:

- **Laboratory Strength Testing** – Data gathered from laboratory testing of material samples to analytically determine compressive, tensile and shear strength properties of individual lithological units;
- **Data from drill core** - Field strength indices, point load indices, weathering indices, defect plane orientation and character and infill type and strength;
- **Surface mapping data** – Data gathered from pit wall exposure mapping (cell / window or scan line mapping), essentially similar to that gathered from drill core.

The most appropriate methods of collecting and interpreting data from each of the above sources will be discussed in detail in the subsequent section, including how this information may be utilised in the formulation of a probabilistic geotechnical model.

2.6 Structural Geology Data Collection

The corner-stone of any practical rock mechanics analysis is the geological model and the geological data base upon which the definition of rock types, structural discontinuities and material properties is based.

Even the most sophisticated analysis can become a meaningless exercise if the geological model upon which it is based is inadequate or inaccurate. Methods for the collection of geological data have not changed a great deal over the past 25 years and there is still no acceptable substitute for the field mapping and core logging. There have been some advances in the equipment used for such logging and a typical example is the electronic compass illustrated in Figure 2.6.1.

The emergence of geological engineering or engineering geology as recognised university degree courses has been an important step in the development of rock engineering.

These courses train geologists to be specialists in the recognition and interpretation of geological information which is significant in engineering design. These geological engineers, following in the tradition started by Stini in the 1920s, play an increasingly important role in modern rock engineering.

Figure 2.6.1 - A Clar electronic geological compass manufactured by F.W. Breihapt in Germany

2.7 Definition of Geological Terms

The following geological terms that will be used in this thesis. They are described below as follows:

- **Intact Rock** – Refers to the consolidated and cemented assemblage of mineral particles which form the intact blocks between discontinuities (geological

structure) in the rock mass. In most hard igneous and metamorphic rocks the strength is one or two orders of magnitude greater than that of the rock mass.

- **Discontinuities** – Refers to planes of weakness or structural features which separate intact rock blocks within a rock mass. These features are sometimes collectively referred to as joints. It should be noted that this is an over simplification as the mechanical properties of these features are related to the process by which they were formed i.e. the Geomorphology.
- **Discontinuity Sets** – Refers to the system of discontinuities which have approximately the same inclination and orientation.
- **Rock Mass** – Refers to the insitu rock which has been rendered discontinuous by systems of structural features such as joints, faults and bedding planes. Slope failure in rock mass is generally associated with movement on these discontinuity surfaces.
- **Continuity / Persistence –** Refers to the length of the respective geological features. Whereby a fault may have a larger trace length say in excess of 15m and a joint may have a trace length of a few meters. The relative lengths of the individual features will provide an indication of the anticipated failure volume.
- **Infill / Gouge** – Refers to the material between two faces of a structural discontinuity such as a fault. This material may be the debris resulting from the sliding of one surface upon another or it may be the material which has been precipitated from solution or caused by weathering.

2.8 Presentation of Geological Data

One of the most important aspects of rock slope analysis is the systematic collection and presentation of geological data in such a way that it can be easily evaluated and incorporated into a stability analysis.

Experience, after Hoek et al (1973) has shown that spherical projections provide a convenient means for the presentation of geological (structural) data. The lower hemispherical equal angle projection technique will be used in this research thesis for the presentation of structural data.

In adapting this projection technique to structural geology, the traces of planes on the surface of the reference sphere are used to define the dips and dip directions of the planes. The data is then represented by plotting radial lines for each feature onto a reference sphere which is fixed in space.

Hence any radial line joining a point on the surface to the centre of the sphere will have a fixed direction in space. If this sphere is now moved so that its centre lies on the plane under construction, the great circle is traced out by the intersection of the plane and the sphere will define uniquely, the inclination and orientation of the plane in space.

Since the same information is given on both the upper and lower parts of the sphere, only once of these need be used. In engineering applications the lower reference hemisphere is used for the presentation of data.

In addition to the great circle, the inclination and orientation of the plane can also be defined by the pole of the plane. The pole is the point at which the surface of the sphere is pierced by the radial line which is normal to the plane.

Once the geological data has been collected, computer processing of this information can be of considerable assistance in plotting the observed traces and in the interpretation of statistically significant trends.

Figure 2.8.1 illustrates a plot of contoured pole concentrations and corresponding great circles produced by the software program DIPS developed at the University of Toronto and now available from Rocscience Inc.

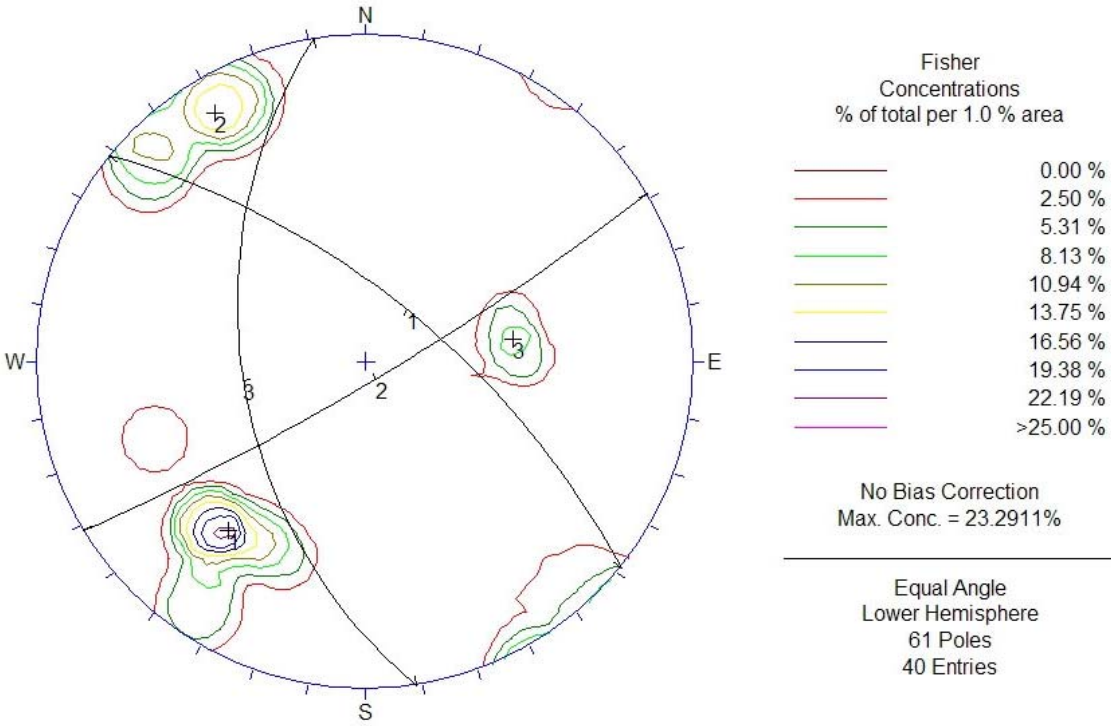


Figure 2.8.1 - Plot of structural features using the program DIPS

Surface and down-hole geophysical tools and devices such as borehole cameras have been available for several years and their reliability and usefulness has gradually improved as electronic components and manufacturing techniques have advanced.

However, current capital and operating costs of these tools are high and these factors, together with uncertainties associated with the interpretation of the information obtained from them, have tended to restrict their use in rock engineering. It is probable that the use of these tools will become more widespread in years to come as further developments occur.

2.9 Laboratory Testing

There has always been a tendency to equate rock mechanics with laboratory testing of rock specimens and hence laboratory testing has played a disproportionately large role in the subject.

This does not imply that laboratory testing is not important but it is the intent of this research thesis to demonstrate that only about ten percent of a well balanced rock mechanics program should be allocated to laboratory testing.

Laboratory testing techniques have been borrowed from civil and mechanical engineering and have remained largely unaltered for the past twenty-five years. An exception has been the development of servo-controlled stiff testing machines which permit the determination of the complete stress-strain curve for rocks.

This information is important in the design of slope excavations since the properties of the failed rock surrounding the excavations have a significant influence upon the stability of the excavations.

Another aspect of laboratory testing is to correlate the empirical strength assessments made from core logging data to actual (analytically assessed) strength values. This is one of the fundamental paradigms of the Rock Mass Variability Index and the basis for this research thesis.

Typically the following information is analysed as part of a laboratory testing programme:

- **Unconfined Compressive Strength (UCS)** testing, to assess the compressive strength of the respective lithological units so as to correlate this information with the empirical strength assessments recorded during the core logging process i.e. from field strength indices or point loads;

- **3 stage direct shear testing,** to assess the shear strength of individual defect planes under various levels of confinement and to correlate empirically recorded Joint Roughness Coefficients (JRC) to shear strength values; and
- **3 stage triaxial testing,** to assess the shear strength of a rock mass under various levels of confinement.

2.10 Core Logging & Empirical Rock Mass Classification

A major deficiency of laboratory testing of rock specimens is that these specimens are usually limited in size and therefore represent a very small and highly selective sample of the overall rock mass from which they were removed.

In a typical engineering project, the samples tested in the laboratory represent only a very small fraction of one percent of the volume of the overall rock mass. In addition, since only those specimens which survive the collection and preparation process are tested, the results of these tests represent a highly biased sample. Hence these results would not provide a reasonable estimate of the properties and more importantly the variability of the in situ rock mass.

It is the intent of this research to provide rock mechanics practitioners' guidance on the selection of properties for the modelling of rock masses, using the available rock mass classification systems have been developed by others.

In Japan, for example, there are 7 rock mass classification systems, each one developed to meet a particular set of needs. Probably the most widely known classifications, at least in the English speaking world, are the Rock Mass Rating (RMR) system of Bieniawski (1976, 1989) and the Q system of Barton, Lien and Lunde (1974).

The classifications include information on the strength of the intact rock material, the spacing, number and surface properties of the structural discontinuities as well as

allowances for the influence of subsurface groundwater, in situ stresses and the orientation and inclination of dominant discontinuities.

These classifications were developed primarily for the estimation of the support requirements in tunnels but their use has been expanded to cover many other fields, including slope stability assessments.

Provided that they are used within the limits for which they were developed, as discussed by Palmstrom and Broch (2006), these rock mass classification systems can be very useful practical engineering tools, not only because they provide a starting point for slope stability assessment but also because they force users to examine the properties of the rock mass in a very systematic manner.

It is recommended that when data is being collected as part of any geotechnical drilling and logging programme, it is done so to facilitate the calculation of one of these empirical rock mass rating systems, for example:

- The **Q system**, after Barton (1974);
- The **Geological Strength Index, (GSI)** system after Hoek (1995);
- The **Rock Mass Rating (RMR)** system, after Bieniawski (1976 & 1989).

The primary reason for this is that it provides a method of numerically assessing the variability of the material i.e. what the effect of an alteration zone or fault zone does to the overall competence of the rock mass.

However it should concurrently be noted that the sole use of these empirical systems for slope design purposes may prove misleading, as it does not consider the effects of structural control, on the large scale i.e. the influence of faults or shear zones.

What needs to be appreciated is that whatever method is chosen, the information should be easily manipulated to assess the variability of intact rock strength and defect plane characteristics of the rock mass in question.

For the purposes of this research thesis the RMR_{89} system will be considered, whereby data gathered from core logging has to be readily converted to obtain RMR_{89} values. To achieve this, the following information was recorded during the core logging process for the respective case studies:

- Core Recovery & Rock Quality Designation (RQD);
- Lithology, degree of weathering, alteration types and strength class;
- Defect (structure) type, number of defect sets & orientation information for the respective structures i.e. alpha and beta angles; and
- Defect plane characteristics (rough, smooth, stepped etc) and infill type and thickness.

This above information can be readily converted to obtain RMR_{89} values using the parameter ratings as shown within the chart below (Table 2.10.1).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low-range uniaxial compressive strength is preferred		
		Uniaxial comp strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5-25 Mpa	1-5 Mpa	<1M Pa
	Rating		15	12	7	4	2	1	1
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	5		
3	Spacing of discontinuities		> 2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	<60 mm		
	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 < 5 mm thick Separation 1.5mm non-continuous	Soft gouge > 5mm thick or Separation >5mm non continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water pressure) / (Major principal σ)	0	0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		
B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)									
Strike and dip orientations			Very 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		20 years for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hours for 2.5 m span	30 mins for 1 m span
Cohesion of rock mass (kPa)		> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)		> 45	35 - 45	25 - 35	15 - 25	< 15
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions						
Discontinuity length (persistence)		< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 30 m
Rating		6	4	2	1	0
Separation (aperture)		None	< 0.1 mm	0.1 - 10 mm	1 - 5 m	< 5 mm
Rating		6	5	4	1	0
Roughness		Very rough	Rough	Slightly rough	Smooth	Slickensided
Rating		6	5	3	1	0
Infill (gouge)		None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
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 TUNNELLING**						
Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		
Drive with dip-Dip 45° - 90°		Drive with dip-Dip 20° - 45°		Dip 45° - 90°		Dip 20° - 45°
Very favourable		Favourable		Very favourable		Fair
Drive against dip-Dip 45° - 90°		Drive against dip-Dip 20° - 45°		Dip 0° - 20° - Irrespective of strike		
Fair		Unfavourable		Fair		

Table 2.10.1 Calculation of RMR_{89}, after Bieniawski

2.11 Surface (Wall) Mapping and Empirical Assessment of Rock Mass Strength

One of the major problems confronting designers of engineering structures in rock is that of estimating the strength of the rock mass. This rock mass is usually made up of an interlocking matrix of discrete blocks, as mentioned in previous sections.

These blocks may have been weathered or altered to varying degrees and the contact surfaces between the blocks may vary from clean and fresh to clay covered and slickensided.

Determination of the strength of an in situ rock mass by laboratory testing is generally not practical. Hence this strength must be estimated from geological observations

and from test results on individual rock pieces or rock surfaces which have been removed from the rock mass.

This question has been discussed extensively by Hoek and Brown (1980) who used the results of theoretical, Hoek (1968) and model studies, Brown (1970), Ladanyi and Archambault, (1970) and the limited amount of available strength data, to develop an empirical failure criterion for jointed rock masses i.e. the Hoek-Brown failure criteria.

Hoek (1983) also proposed that the rock mass classification system of Bieniawski could be used for estimating the rock mass constants required for this empirical failure criterion.

This classification proved to be adequate for better quality rock masses but it soon became obvious that a new classification was required for the very weak tectonically disturbed rock masses associated with the major mountain chains of the Alps, the Himalayas and the Andes.

The Geological Strength Index (GSI) was introduced by Hoek in 1994 and this Index was subsequently modified and expanded as experience was gained on its application to practical rock engineering problems. Marinos and Hoek (2000, 2001) published the chart reproduced in Figure 2.11.1 for use in estimating the properties of heterogeneous rock masses such as flysch.

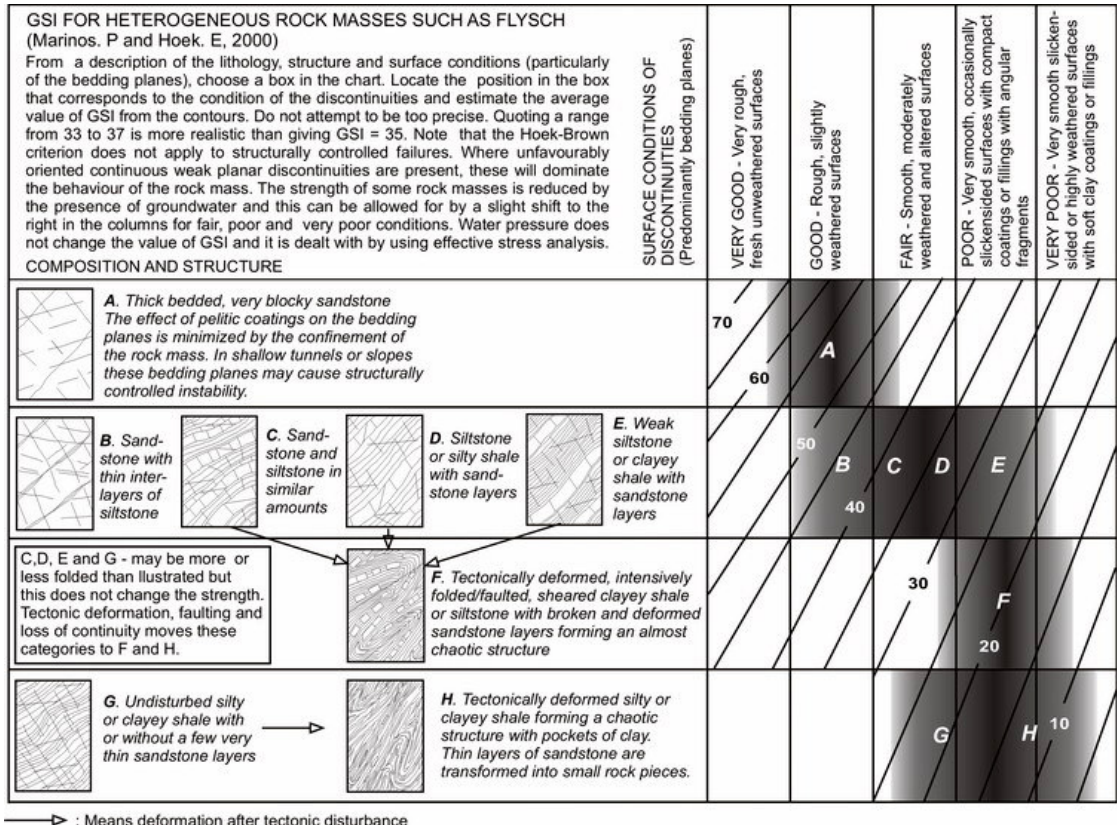


Figure 2.11.1 - Geological Strength Index after Hoek 2000

Figure 2.11.2 - Various grades of flysch in the Pindos Mountains of northern Greece

Practical application of the GSI system and the Hoek-Brown failure criterion in a number of engineering projects around the world have shown that the system gives reasonable estimates of the strength of a wide variety of rock masses.

These estimates have to be refined and adjusted for individual conditions, usually based upon back analysis of slope deformation behaviour, but in general they provide a sound basis for design analyses.

The most recent version of the Hoek-Brown criterion has been published by Hoek, Carranza-Torres and Corkum (2002) and this paper, together with a program called RocLab for implementing the criterion, can be downloaded from the Internet at **www.rocscience.com.**

One of the main drawbacks of the core logging data is that although it records detailed geotechnical information about a rock mass and geological structure, which can be used to assess the variability of structural orientation and defect plane character. It does not assess its performance within a slope. Consequently it becomes difficult to assess how the rock mass will perform under blasting, exfoliation due to over burden removal or to assess the degree of mechanical inter-locking (on an overall slope scale) of the geological structures.

The information recorded during surface mapping shares some similarities with core logging; however there are some key differences particularly in relation to geological structure and slope performance assessment.

Specifically the following 'extra bits' of information are recorded:

- Top and bottom termination observations of an observed geological structure, as well as its lateral (along strike of the slope) persistence so as to assess its influence within a slope exposure, as persistence is a 3D issue not 1D;
- Actual dip and dip direction as mapped from the observed structural plane within a slope as opposed to recording alpha and beta values from drill core that need to be converted to dip and dip direction based on the survey of the drill-hole This usually introduces some level of uncertainty (i.e. survey errors etc).
- Blast damage and degree of exfoliation of structures, i.e. to what level partially or weakly healed features may 'open' up due to rock mass exfoliation from overburden removal.

2.12 Formulation of a Probabilistic Geotechnical Model

One of the key items to that should be appreciated, is that probabilistic slope design potentially enables the slope engineer to design slopes with Factors of Safety (FOS) less than the usual 1.2 or even 1.0, providing the 'consequences' or risk can be catered for.

To put this into context, i.e. how an engineer might recommend a slope with a FOS of less than 1.0 is acceptable, the process by which FOS is calculated needs to be understood.

FOS is a ratio of the resisting forces against the driving forces i.e.

$$FOS = \frac{\sum \textit{Resisting Forces}}{\sum \textit{Driving Forces}} \quad \text{- Equation 2.1}$$

Where:

Resisting Forces

- The rock mass strength;
- Cable Bolts, Shear Pins, (any form of mechanical ground support)
- Gabion blocks, rock fill buttress etc.

Driving Forces (Destabilising Forces)

- Rock mass over burden (gravity stresses);
- Groundwater / pore pressures;
- Seismic forces;
- External loading i.e. crusher / process plants, waste dumps etc.

What does a FOS of less than 1.0 indicate? Essentially it indicates that the sum of the driving forces is greater than the sum of the resisting forces. This implies that the

slope is continually displacing, i.e. moving. It has been demonstrated that it is quite tolerable for large displacements to take place and the slope still remains serviceable.

The serviceability criteria should be taken into context, whereby a mine pit slope well away from any infrastructure may be able to sustain large amounts of displacement, in contrast to a slope in close proximity to a major highway or public infrastructure, which may not be able to tolerate large amounts of displacement.

The formulation of a probabilistic geotechnical model for stability analysis involves combining and comparing the respective distributions from data assessed empirically as part of core logging and surface (wall) mapping, and analytically from laboratory testing.

In particular the following components of the geotechnical slope model require definition:

- **Rock mass compressive and shear strengths** for the individual lithological units within a slope sector, including alteration zones;
- **Geological structure i.e. individual defect types**, their orientation and mechanical characteristics within the respective lithological units and alteration zones; and
- **Insitu Hydro-geological conditions** within the respective slope design sectors and its interaction with the insitu rock mass and geological structures.

2.13 Rock Mass Failure Criterion

Intact rock has both tensile and compressive strength, but the compressive to tensile strength ratio is quite high, generally about 20. In uniaxial tension, failure follows the maximum principal stress theory, refer to Figures below.

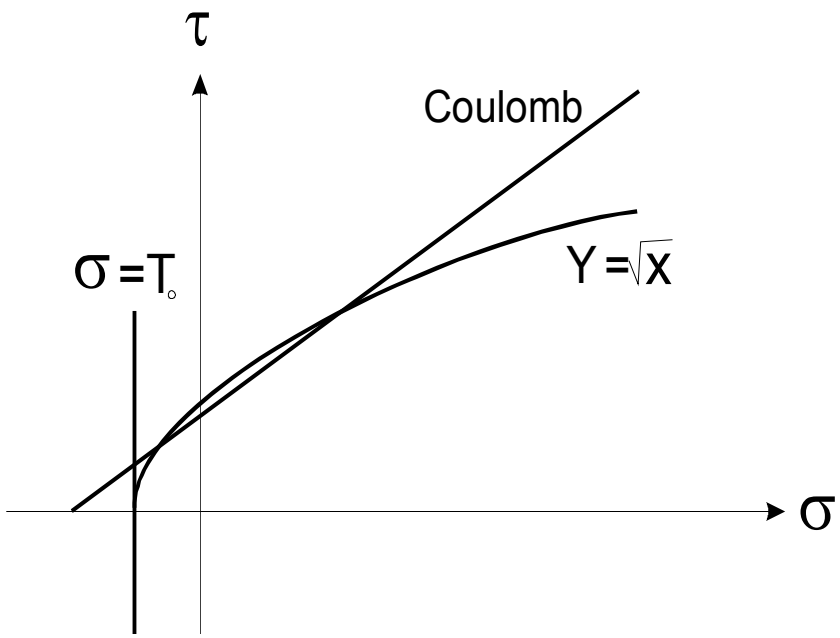
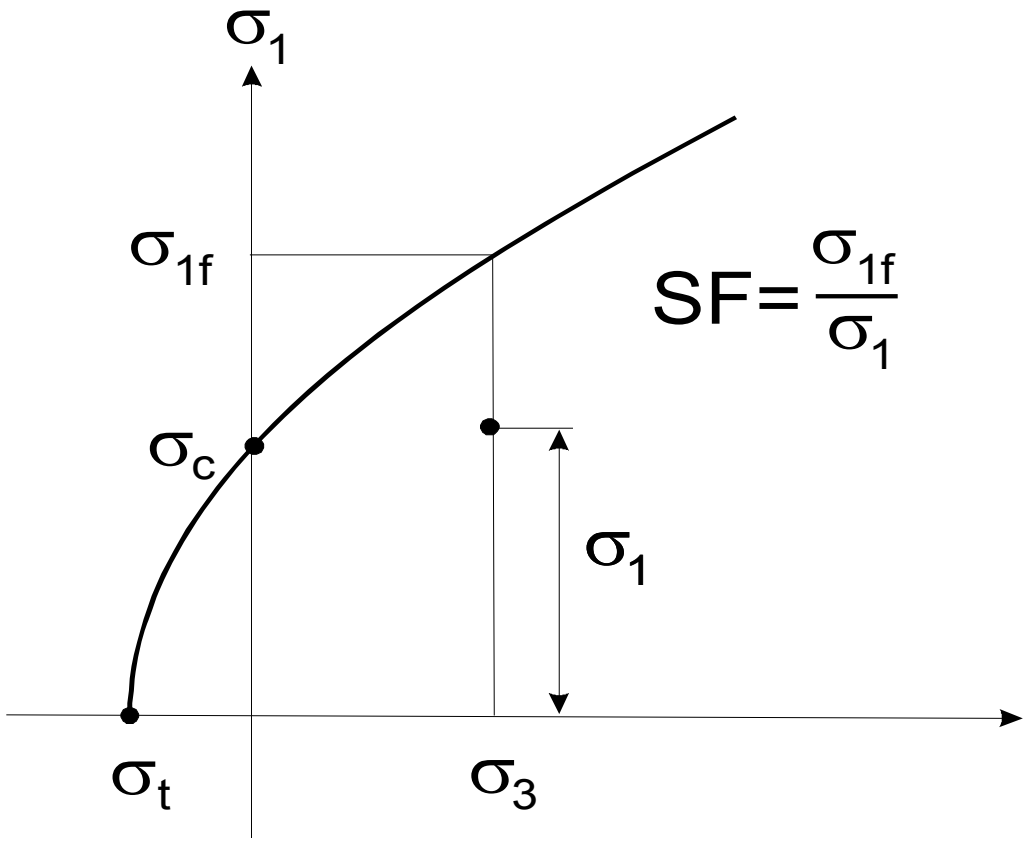


Figure 2.13.1- Failure criteria in Mohr's diagram



$$\sigma_3 = T_o$$

Figure 2.13.2 - The Hoek and Brown rock strength function

Scrutiny of the above figures suggests that the other two principal stresses have no influence on failure. At failure a fracture plane forms that is oriented perpendicular to the σ_3 (Figure 2.13.2). Note that the Coulomb theory would predict shear failure in uniaxial tension at 45- σ_2 with σ_3.

There was a suggestion to combine the Coulomb theory with the maximum stress theory (the tension cut-off) which would predict the proper orientation of the failure plane for both tension and compression.

Others (Hoek et al) would rather replace both with a $y^2=x$ type of parabola (Figure 2.13.2). As the shear fracture does not appear at point of failure, this aspect of the Coulomb theory is considered meaningless.

In fact, there is little point in using the Mohr's diagram in rock mechanics, failure conditions are more meaningfully presented in the σ_3- σ_1 space using a nonlinear function for strength. Although there are many variations of this function, the most popular one is due to Hoek and Brown (1980) which has the general form of:

$$\sigma_{1f} = \sigma_3 + \sqrt{\sigma_3 \sigma_c m + s\sigma_c^2}$$

- Equation 2.2

Where σ_c is the uniaxial compressive strength of the intact rock, m is a constant (characteristic of the rock type) and s is a rock mass parameter. s=1 for intact rock. Typical values of the m parameter can be found in the first row of Table 2.13.1

The s parameter is significant only in extending the strength function to the strength of the rock mass. The same diagram is often used to define the safety factor for an existing state of stress (σ_3, σ_1).

Representative rocks		m_i [-]
Limestone rocks with well developed crystalline cleavage	Dolomite, calcite, marble	≈ 7
Consolidated clayey rocks	Mudstone, siltstone, silty shale, slate	≈ 10
Sandy rocks with solid crystals and poorly developed crystalline cleavage	Sandstone, quarzite	≈ 15
Fine grained igneous crystalline rocks	Andesite, dolerite, diabase, rhyolite	≈ 17
Coarse grained and metamorphic rocks	Amphibolite, gabbro, gneiss, granite, diorite	≈ 25

Table 2.13.1 – Typical m Vales for Hoek - Brown Failure Criterion

2.14 Assessment of Rock Mass Compressive Strength

Rock mass compressive strength can be assessed empirically from field strength and point load indices recorded during core logging or surface mapping, typically (assuming the RMR_{89} system) the following ranges of strength values are recorded.

Strength Description	Estimated UCS Value
Extremely Weak (EW)	0.25MPa
Very Weak (VW)	1.75MPa
Weak (W)	6.5MPa
Moderately Strong (MS)	20MPa
Strong (S)	40MPa
Very Strong (VS)	100MPa
Extremely Strong (ES)	>250MPa

Table 2.14.1 Strength Descriptions Recorded During Core Logging (ISRM)

This information can be plotted as a histogram to assess the distribution of the data. A typical plot of field strength index data from core logging is shown below.

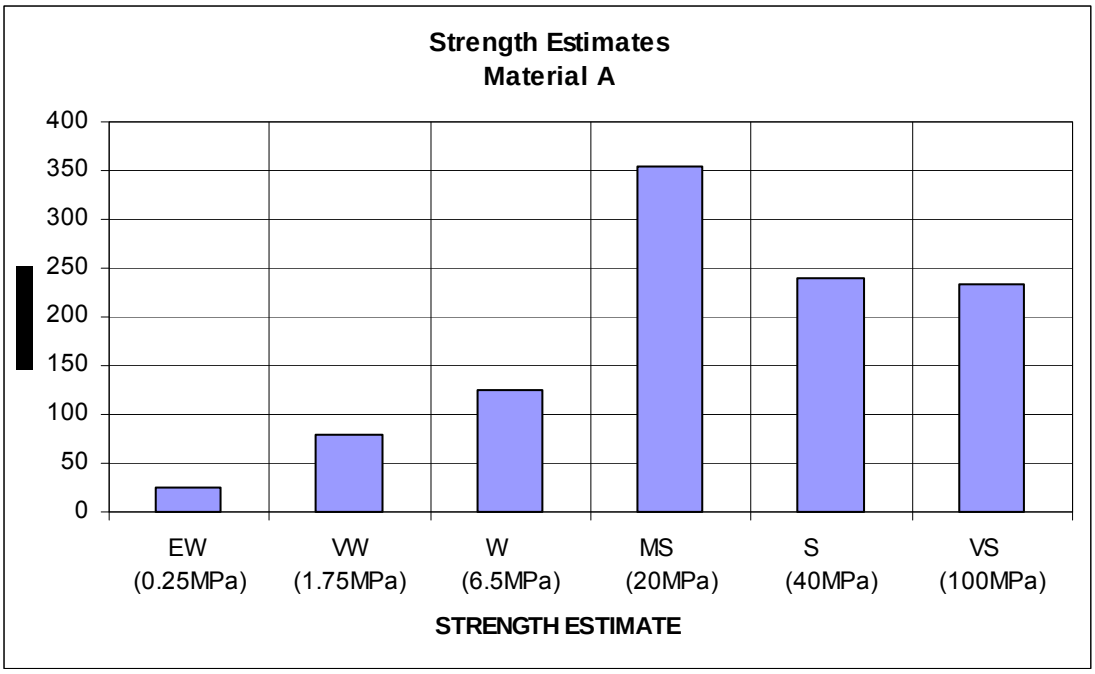


Figure 2.14.1- Histogram of Strength Estimates for Material A from Core Logging

Based on this distribution the following statistical information can be derived as tabulated below.

Mean (M)	Mode (Mo)	Median (Md)	Standard Deviation (S)	Coefficient of Variability (S/M)
38.8MPa	20.0 MPa	20.0 MPa	34.8 MPa	89.7%

Table 2.14.2 - Statistical Parameters of Material A's Strength Estimates

Figure 2.14.2 below shows a histogram of the distribution of UCS values for this material, as obtained from laboratory testing. It can be seen from the frequencies of each 'bin' that the total number of samples tested to assess UCS is significantly less than the number of available field strength indices. This implies that the distribution of field indices would provide a more accurate representation (i.e. more representative of insitu conditions) of this material's variability.

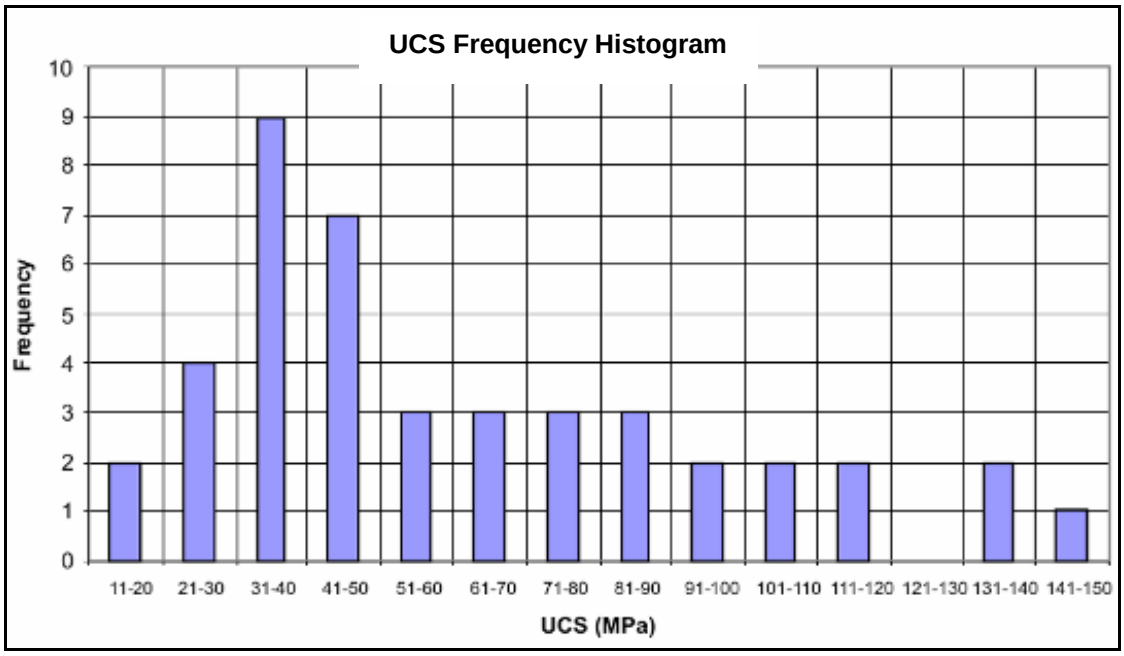


Figure 2.14.2 - UCS Distribution for Material A

Mean (M)	Mode (Mo)	Median (Md)	Standard Deviation (S)	Coefficient of Variability (S/M)
61.6MPa	48.6MPa	49.8MPa	34.4MPa	55.8%

Table 2.14.3 Statistical Parameters of Material A's UCS Test Results

How does one choose a design value and assign variability i.e. choose a Coefficient of Variability (COV) for input into a stability model for the purposes of a statistical analysis?

Typically when conducting deterministic modelling the mean value from UCS testing would be chosen, in this case it would be 61.6 MPa. If this was done the 'lower bound' UCS or field strength estimates end up being excluded. This may result in an optimistic representation of stability conditions, which may not prove achievable in reality.

For example, the lower material strength values may represent an alteration zone that could potentially initiate failures affecting multiple batters, which could be further exacerbated by the presence of continuous geological structure that could even result in overall slope instability.

An important point to note is that some of the weaker materials (say alteration zones within a particular lithological unit) may not get tested in some instances as they are destroyed either during transportation or during manual handling.

Hence to obtain an indication of the 'lower bound' strength values the field strength indexes should be considered along with their respective variabilities.

As part of the methodology presented in this research thesis, the Rock Mass Variability Index, tactic proposes choosing a mean value for compressive strength from the UCS test results and assigning the coefficient of variability as obtained from the field strength indices. This would ensure that a realistic strength value is utilised within the model and the entire variability (as far as sampled) is considered within it.

2.15 Assessment of Rock Mass Shear Strength

The most important parameter that requires careful assessment when designing open pit mines is the rock mass shear strength components i.e. cohesion and friction, as typically instabilities initiate at low levels of confinement (normal stress) and result in shear failure of the rock mass.

Designing structures in rocks and rock masses bears many similarities to techniques that have been developed for soils. There is however a number of major differences:

(1) The scale effect is overwhelming in rocks. Rock strength varies widely with sample size. At one end, we have the intact rock (homogenous, isotropic, solid, and continuous with no obvious structural defects) which really exists only at the hand-specimen scale. At the other end is the rock mass that is heterogeneous and anisotropic carrying all the defects that is characteristic of the rock mass at the field scale. In the design of engineering structures in rock, the size of interest is determined by the size of the rock mass that carries the stresses that are imposed upon it.

(2) Rock has tensile strength. It may have substantial tensile strength at the intact rock scale, but much smaller at the scale of the rock mass. Even then only in exceptional circumstances can the rock mass be considered as a “tensionless” material. Intact rock fails in tension along planes that are perpendicular to maximum tension (or minimum compression) and not along shear planes as suggested by the Coulomb theory, and discussed above.

(3) The effect of water on the rock mass is more complex.

(a) Pore space in most intact rocks is very small and so is the permeability. The water contained in the pore space is not necessarily free water. The truly free water exists only in the rock mass, in fractures, where water may flow at high rates.

(b) In contrast to soils, water is more compressible (by about one order of magnitude) than intact rock. The difference would be smaller when compared with the compressibility of the rock mass especially close to the free surface where loose rock is commonly found.

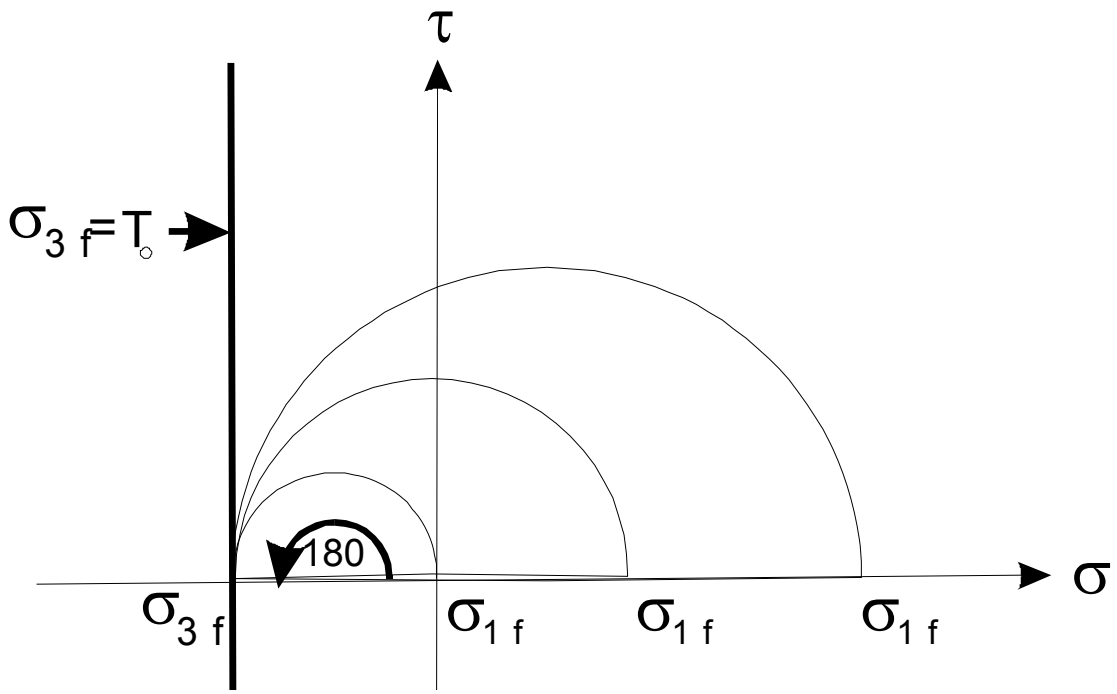


Figure 2.15.1 - The maximum stress theory(Tensile Strength)

The strength of the rock mass is only a fraction of the strength of the intact strength. The reason for this is that failure in the rock mass is a combination of both intact rock strength and separation or sliding along discontinuities. The latter process usually dominates. Sliding on discontinuities occurs against the cohesional and/or frictional resistance along the discontinuity.

The cohesional component is only a very small fraction of the cohesion of the intact rock. In designing with the rock mass, two different procedures are used. When a rock block is well defined, its stability is best evaluated through a standard “rigid-body” analysis technique.

All the forces on the block are vector-summed and the resultant is resolved into tangential and normal components with respect to the sliding plane. The safety factor becomes the ratio of the available resistance to sliding to the tangential (driving) force. This is the technique used in slope stability analysis.

The second technique is stress rather than force-based. Here the stresses are evaluated (usually modelled through numerical procedures) and compared with available strength. The latter is expressed in terms of the Hoek and Brown or Mohr-Coulomb rock mass strength function. This is where the s parameter becomes useful. $s=1$ for intact rock and $s<1$ for the rock mass.

Rock mass shear strength can be determined empirically using correlations between RMR_{89} values and shear strength components after Bieniawski (1989), and analytically using three stage triaxial testing of material samples.

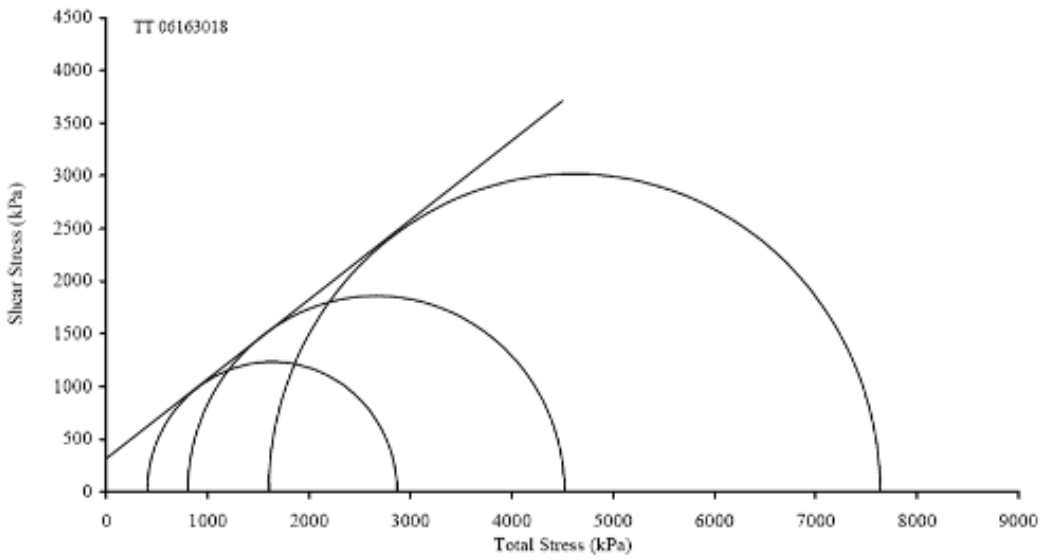
As mentioned in earlier sections of this thesis geotechnical data should be gathered in a manner so as to facilitate the calculation of RMR_{89} or some other form of empirical rock mass rating system. As it is an ideal method to assess the variability of the overall rock mass as it includes the effects of structure and alteration.

It should be noted that when determining rock mass shear strength based solely on the RMR_{89} values, it may provide inaccurate or dubious results as they are 'experience' based correlations as opposed to being site specific. However the distribution of RMR_{89} values for a particular lithological unit is an ideal method to calculate the relative variability of the material.

What the slope design engineer should do is rely on the tri-axial results to provide the actual (analytical) values of cohesion and friction angles as interpreted from the stress path fitted to the Mohr's envelopes produced from three stage tri-axial testing, and assign variabilities to the individual parameters based on the COV of the RMR_{89} distribution. An example is discussed below.

Figure 2.15.2 shows the Mohr's circles (envelopes) and a fitted stress path as derived from a three stage tri-axial test, from which cohesion and friction angle values were interpreted. In this case 320 kPa and 37° have been interpreted as the cohesion and friction angle respectively.

Shown in Table 2.15.1 is a statistical summary of the for the RMR_{89} values for this lithological unit. Figure 2.15.2 is the distribution of the RMR_{89} values.



SHEAR STAGE DATA

Stage No	Strain Rate (mm/min)	ε_f (%)	σ_3 (kPa)	$(\sigma_1)_f$ (kPa)	$(\sigma_1 - \sigma_3)_f$ (kPa)
1	0.015	0.45	400	2867	2467
2	0.015	0.37	800	4520	3720
3	0.015	0.35	1600	7634	6034

Cohesion, C (kPa) : 320 **Friction Angle, Ø** (degrees) : 37

Figure 2.15.2 - Mohr's Circles from a 3 Stage Triaxial Test

Mean (M)	Mode (Mo)	Median (Md)	Standard Deviation (S)	Coefficient of Variability (S/M)
46.7	42.0	47.0	8.8	18.8%

Table 2.15.1 Statistical Parameters of Material A's RMR Values

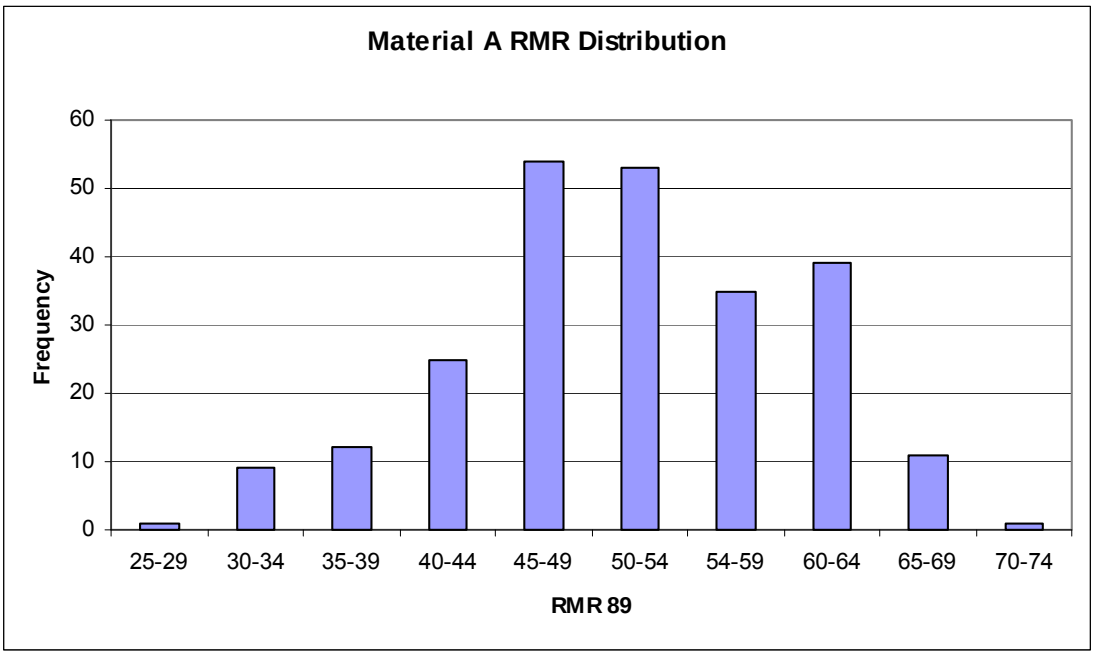


Figure 2.15.3 - RMR Distribution for Material A

As three stage tri-axial testing is relatively expensive and time consuming to conduct many are usually done as part of a typical rock mechanics investigation. Therefore due to the relatively sparse data set of tri-axial values it is usually difficult to obtain meaningful statistical distributions of rock mass shear strengths.

However by obtaining the material variability distribution from the RMR_{89} values, these can be combined with the laboratory data (i.e. from tri-axial testing) in a similar manner to which the UCS values were combined with the COV calculated from the field strength indices.

Hence in the above case the slope design engineer would choose average values of cohesion and friction angle of 320kPa and 37°, and assign a COV of 18.8 percent to both of these values. Figures 2.15.4a and 2.15.4b depict the distribution of these components as will be modelled with a stability analysis package i.e. Slide or Phases.

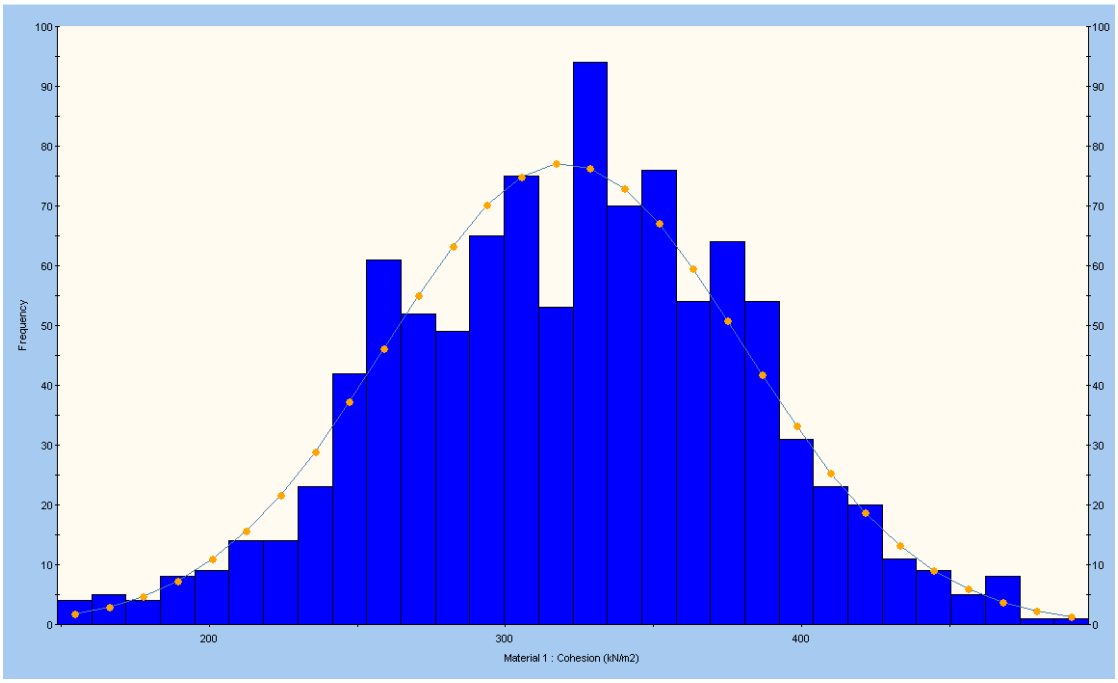


Figure 2.15.4a - Distribution of Cohesion Values

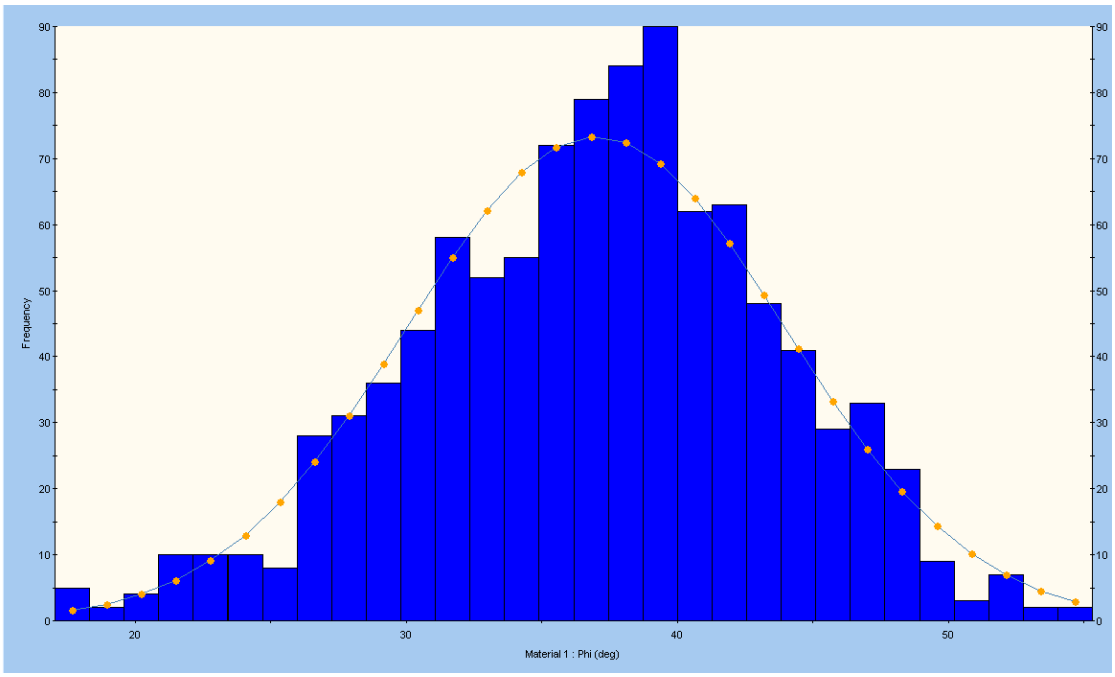


Figure 2.15.4b Distribution of Friction Angle Values

A point to note is that the RMR_{89} values are calculated over intervals as opposed to points hence the recording of this information is at the discretion of the geotechnical

logger to identify zones of alteration or changes in structural regime etc. Therefore it is important that the logging is undertaken by an experienced geologist or engineer.

2.16 Geological Structures

In most open pit iron ore mines the batter (bench) geometry is strongly influenced by geological structures, (as the mineralisation is typically hosted within a bedded sequence), except in the presence of very weak rock masses / alteration zones or excessive groundwater pressures (pore pressures).

The orientation of geological structure is typically assessed from surveyed drill core i.e. by recording alpha and beta angles. One of the draw backs of this is that it is difficult to determine the relative persistence (trace lengths) of the individual features and hence the level of mechanical inter-locking they exhibit within the slope.

It is common practise for slope design engineers to perform kinematic (stereographic) analyses, based on contoured structural sets obtained solely from geotechnical drilling and core logging to assess the likelihood of structurally controlled slope instability mechanisms.

A downside of performing this analysis on a stereonet is that the persistence of a particular structure is not taken into account. An example is shown in Figure 2.16.1 below where the two structures shown as dashed red lines would form a wedge in ‘space’ i.e. on a stereonet, but in reality (within the slope) they do not. This is due to the limited persistence of the respective features; they are not ‘long’ enough to intersect each other to form the sliding wedge.

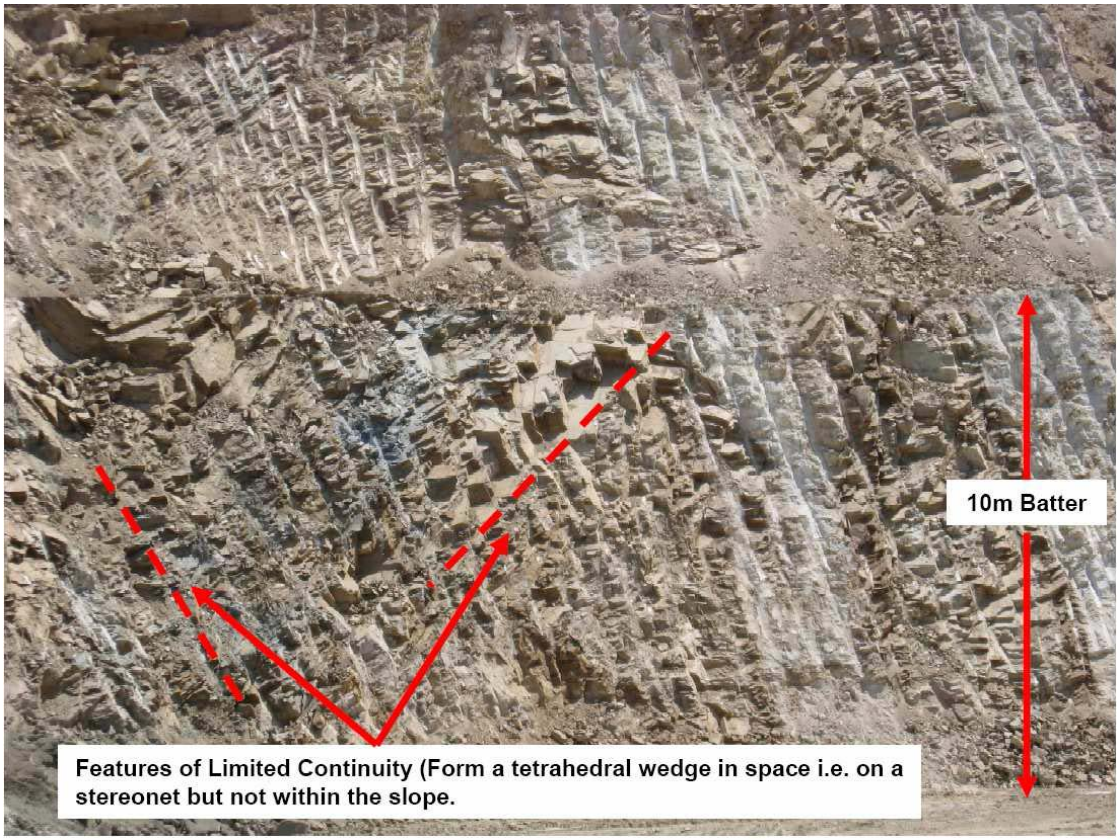


Figure 2.16.1 Pit Wall Exposure

The stereonet below depicts the structures as mapped from the exposure shown in Figure 2.16.1, it can be seen that for this particular slope, there are a number of structures (poles) that fall within the daylight envelope of the slope face, which should in theory result in planar failures and where they intersect appropriately orientated structures, tetrahedral wedge failures are feasible as well.

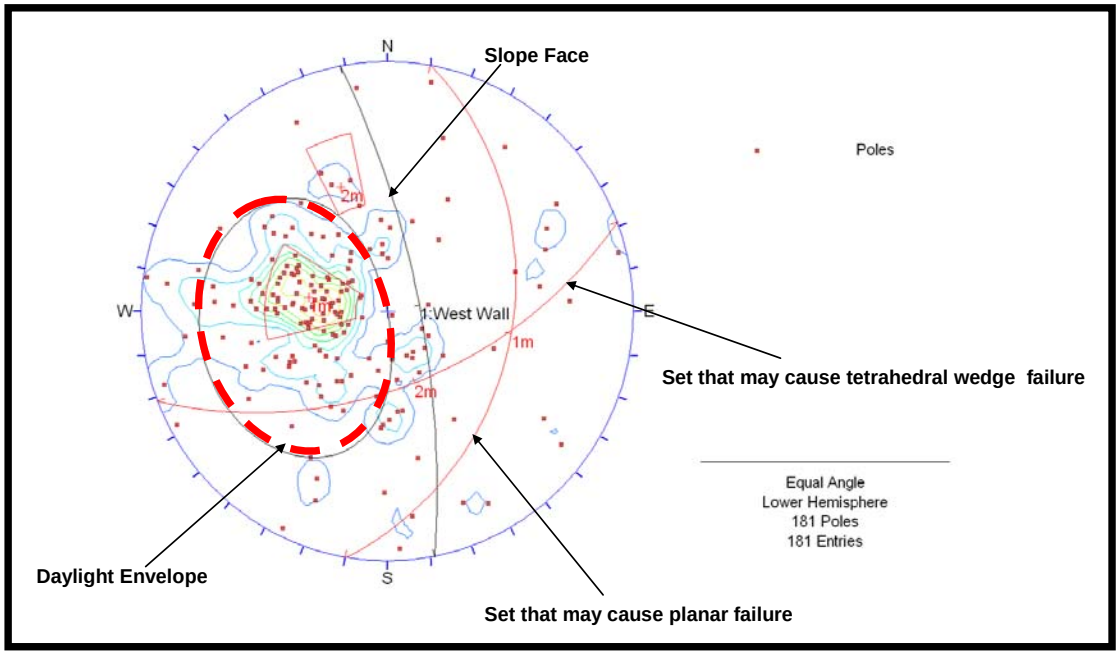


Figure 2.16.2 Stereonet of a Slope Face

However, as stereonets do not consider the persistence of the structures the likelihood of these failures occurring (i.e. probability of the trace lengths of these structures are long enough to intersect each other) cannot be accurately assessed.

For example consider a batter wall height of 20m, a structure that has a 1m trace length will not result in overall batter slope instability. It may however result in the formation of small scale blocks that can be operationally managed via batter wall clean ups and scaling.

Hence if the persistence of structures was recorded, the data plotted within the stereonet could be filtered to depict structures of 'importance' i.e. those that have a continuity of half a batter wall height or greater.

It should be noted that the assumption is being made that failures that affect less than half a batter can be operationally managed; however this is dependent on available mining equipment and operational constraints, which will vary from site to site.

The histogram below shows the distribution of the trace lengths of the individual features that fall within the daylight envelope of the slope from Figure 2.16.1. It can be seen that of the structures that fall within the daylight envelope only 27 features have a trace length greater than 10m.

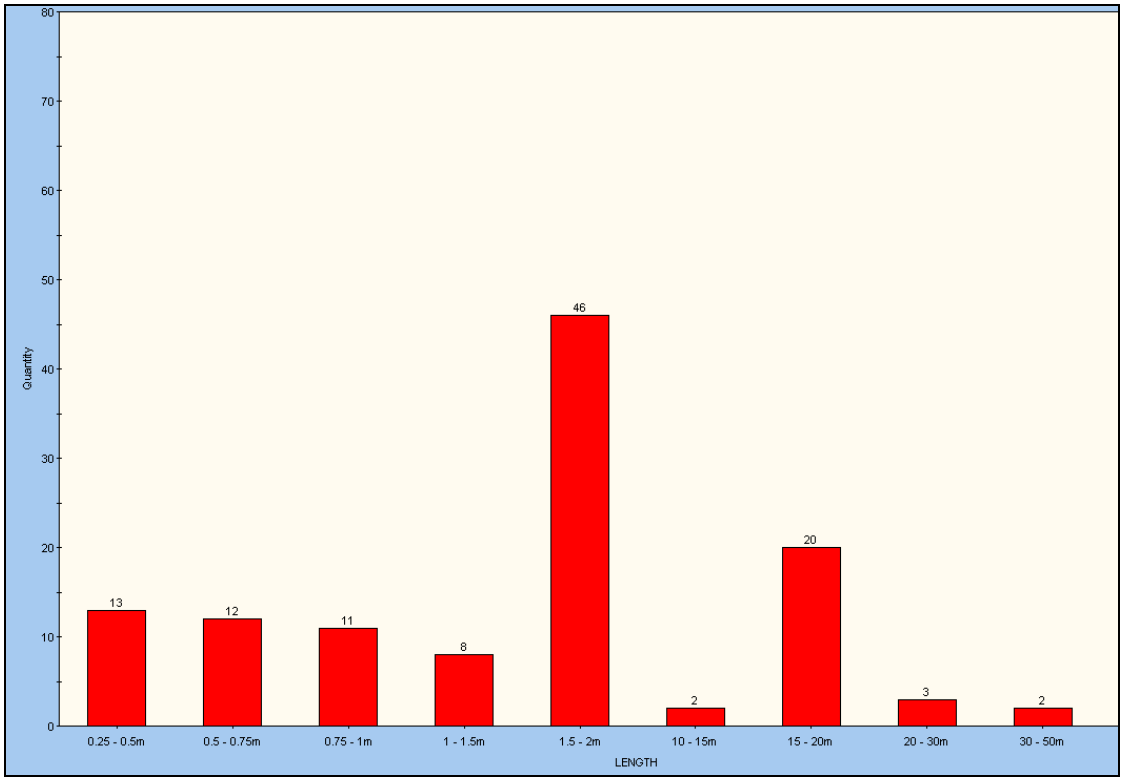


Figure 2.16.3 Distribution of Trace Lengths of Structures

Hence when calculating the likelihood (probability) of planar or tetrahedral wedge sliding the engineer needs to consider the following:

- What is the probability that a given structure will fall within the daylight envelope of a particular slope, P_{DE}.
- What is the probability that a structure will have a persistence of half the batter height or greater, $P_{0.5BH}$.

- Sliding or failure will occur when there are structures that satisfy both criteria's i.e. that fall within the daylight envelope of the slope and have a trace length of greater than half the batter height,

I.e. $P_{Failure} = P_{DE} \times P_{0.5BH}$ *- Equation 2.3*

2.17 Hydrological Conditions

Groundwater pressures are a major factor in all slope stability problems and an understanding of the role of subsurface groundwater is an essential requirement for any meaningful slope design, Hoek and Bray (1981), Brown, (1982).

While the actual distributions of water pressures in rock slopes are probably much more complex than the simple distributions normally assumed in slope stability analyses, Freeze and Cherry (1979), sensitivity studies based upon these simple assumptions are generally adequate for the design of drainage systems, Masur and Kaufman (1962).

Monitoring of groundwater pressures by means of piezometers, Brown (1982) is the most reliable means of establishing the input parameters for these groundwater models and for checking the effectiveness of drainage measures.

In the case of dams, forces generated by the water acting on the upstream face of the dam and water pressures generated in the foundations are critical in the assessment of the stability of the dam.

Estimates of the water pressure distribution in the foundations and of the influence of grout and drainage curtains upon this distribution have to be made with care since they have a significant impact upon the overall dam and foundation design, Soos (1979).

The major advances that have been made in the groundwater field during the past decades have been in the understanding of the transport of pollutants by

groundwater. Because of the urgency associated with nuclear and toxic waste disposal in industrialised countries, there has been a concentration of research effort in this field and advances have been impressive.

The results of this research do not have a direct impact on conventional geotechnical engineering but there have been many indirect benefits from the development of instrumentation and computer software which can be applied to both waste disposal and geotechnical problems.

2.18 Interaction of Geological Structure and Ground in Rock Slopes

Section 2.16 discussed the importance of the relative persistence of geological structure and the likelihood of failure. This section examines the stability of a block of rock found on a slope, taking into account the influence of geological structure and ground water. Although e a relatively simple problem, it will still illustrate the procedure involved in analysing rock slope stability.

Particular attention should be paid to how the effect of ground water is incorporated into the stability analysis. Typically block analysis is used when it is expected for the block to slide on a single or a combination of discontinuities and there is good control over the geometry.

This implies that the size and hence the weight of the block and the geometry of the slope is known. In the simple two-dimensional case, which is going to be discussed, the geometry of the failure plane is simply the slope angle.

A major problem is how to get a reasonable estimate of the resistance to sliding. In this regard, conditions are similar to rock mass analysis, where we had to come up with an estimate of the rock mass strength. Again, some degree of judgement will have to be applied. There are two ways to proceed:

One is to accept the definition of shear resistance as in the Coulomb theory. This implies that the discontinuity shear strength is made up of two components, a cohesion and a frictional resistance.

The cohesion represents the strength of "solid rock bridges" that may exist at the base of the failure plane which will have to be sheared off to let the block move. This is the most challenging part to estimate, because it may vary between zero and the strength of the solid rock.

Usually, it is a very small fraction of the solid strength. The frictional part is simply the normal force multiplied by the tangent of the friction angle. Forces rather than stresses are used here, and the resistance force according to the Coulomb specification becomes:

$$Discontinuity\ shear\ strength = Cohesive\ force + N \tan \phi$$

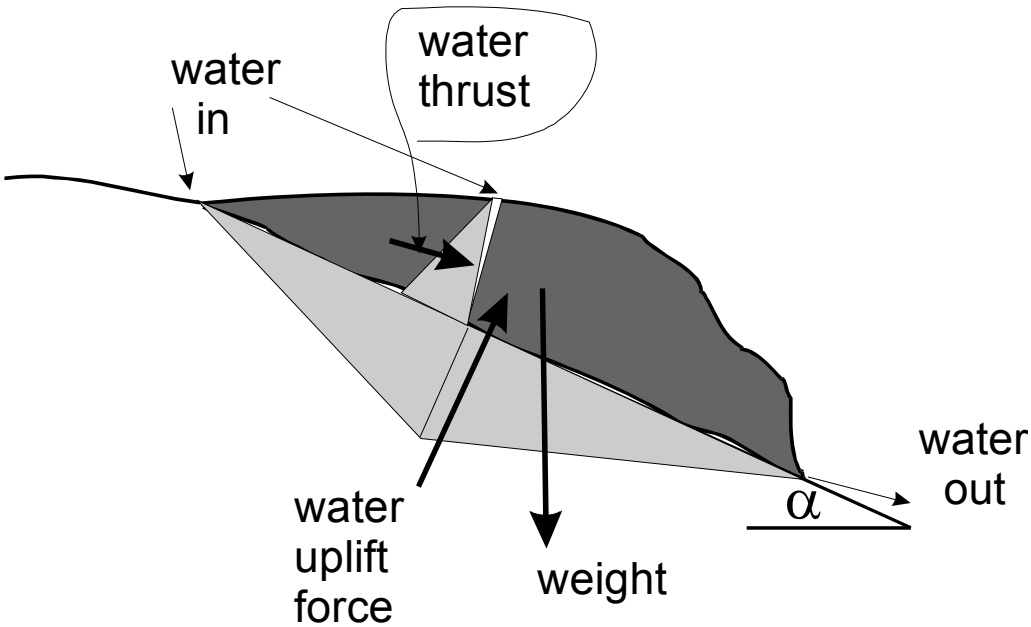


Figure 2.18.1 - Two-dimensional analysis of rock slope stability

The Coulomb type of specification is useful primarily in the "back analysis" of slope failures. Many slope design engineers are familiar with the common chore of

redesigning slopes that have either failed or showed signs of instability (tension crack at the back of the slope). In cases like this, the safety factor should be very close to unity (1.0).

If a good estimate of the friction angle and the loading condition is known then by back-calculating the value of the discontinuity shear strength can be ascertained. As a result of this, the slope can be redesigned to a new safety factor with a lot more confidence.

When back analysis is not possible, a reliable estimate of the available cohesive force is practically impossible. Most design situations fall into this category. What to do then?

There is an alternative way of estimating discontinuity strength proposed by the same author who was involved in constructing the Norwegian Geological Index (NGI) classification (Barton). The Coulomb theory proposes a linear law for discontinuity strength; the Barton specification advances a non-linear law:

$$\tau = \sigma_n \tan\left(JRC \log_{10}\left(\frac{JCS}{\sigma_n} \right) + \phi_b \right)$$ *- Equation 2.4*

Here stress rather than force units are used. σn and τ would refer to the average normal stress and the unit shear strength respectively. For comparison with the Coulomb specification, τ and σ_n are obtained by dividing the shear resistance force and the normal force by the area of contact.

The Barton strength criterion uses three material parameters:

- JRC (joint roughness coefficient), JRC varies between 0 (very smooth, planar joint) and 20 (rough undulating surface);
- JCS (joint compressive strength) and

- ϕ_b (basic friction angle).

JCS is a fraction of the compressive strength of the rock. The compressive strength should be discounted depending on the condition of the rock walls on the two sides of the joint. Usually where the surface is weathered and altered and may carry soft in-filling i.e. fault gauge.

In the latter case, the strength would be very small indeed. The basic friction angle is what is normally call the friction angle determined on a flat surface rubbing against another flat surface of the same rock i.e. residual friction angle.

Apart from needing three parameters as opposed to Coulomb's two, the nonlinear strength is different from the Coulomb law in that it has no strength at zero normal stress. Essentially, the Barton specification is defined in terms of a friction angle that is adjusted for joint roughness and the strength of rock.

In general this method of determining discontinuity shear strength would be adequate when attempting to analyse the safety factor for the sliding block as portrayed in Figure 2.18.1 as follows:

- There is the possibility of sliding down along joint plane sloping at angle α. First, the forces that act on this block of rock should be established. Weight is an obvious one. The water forces are based on the assumption that water flows along the slide plane and perhaps along other joints or as in this case in a tension crack as well.
- If there was no tension crack, an uplift force would be present, arising from the fact that water would normally flow in at the high-elevation end and flow out at the low elevation. The head of water at the intake and discharge points is zero.

- Here a triangular distribution is assumed, where the maximum head occurs at midpoint and its value is one half of the elevation difference between intake and the discharge points.
- The uplift force itself is equal to the area of the pressure distribution diagram (light-shaded area) and acts perpendicular to the slide surface. With a tension crack, there could be slope-parallel water thrust due to water accumulating in the tension crack.
- Its value would be calculated from the upper (small) light-shaded triangle. For this the maximum head would occur at the base, with the maximum head being equal to the elevation difference between the top and the bottom of the tension crack.
- With the loads defined, an estimate of the shear resistance that could develop along the sliding surface can then be determined.

Summarised below are the points mentioned in this section in relation to calculating the factor of safety of the potentially sliding block on a plane:

(1) Determine the elevation difference between the intake and discharge points and the estimate the slope angle. Find the weight of the block of rock using a width of 1 m in the third direction. Assume a reasonable unit weight for the material;

(2) Compute the uplift force and the water thrust;

(3) Resolve all the forces into components, normal and parallel with the slide plane;

(4) Sum the parallel (tangential) forces to get the Driving Force;

(5) Sum the normal forces and get the total frictional resistance by multiplying it with tan ϕ (use the residual friction angle);

(6) Define the cohesive force as unit cohesion times the total area of contact; the unit cohesion will stay as a variable;

(7) Add the cohesive force to the total frictional force;

(8) Formulate the safety factor, equate it with 1 and compute (back calculate) the friction angle; after this operation, all the strength parameters are defined.

(9) Now the true safety factor for the slope can be calculated using the actual cohesion and friction parameters.

2.19 Slope Stability Analysis Methods and Software

Analytical models have always played an important role in rock mechanics. The earliest models date back to closed form solutions such as that for calculating the stresses surrounding a circular hole in a stressed plate published by Kirsch in 1898.

The development of the computer in the early 1960s made possible the use of iterative numerical techniques such as finite element, Clough (1960), boundary element, Crouch and Starfield (1983), discrete element, Cundall (1971) and combinations of these methods, Von Kimmelmann et al (1984), Lorig and Brady, (1984). These have become almost universal tools in rock mechanics.

Today there are a vast range of methods for the stability analysis of rock and mixed rock-soil slopes; these range from simple infinite slope and planar failure limit equilibrium techniques to sophisticated coupled finite-/distinct element codes.

It is less than 25 years since most rock slope stability calculations were performed either graphically or using a hand-held calculator, the exception being advanced analyses involving critical surface searching routines performed on a mainframe computer and Fortran cards. The great majority of early stability analysis programs were in-house with very little software being available commercially.

With access to a personal computer it is possible undertake with relative ease complex numerical analyses of rock slopes. The computer has also made it much more convenient to use powerful limit equilibrium methods, Sarma, (1979) Brown and Ferguson, (1979), Shi and Goodman, (1981), Warburton, (1981) and probabilistic

approaches, McMahon (1971), Morriss and Stoter, (1983), Priest and Brown, (1982), Read and Lye, (1983) for rock mechanics studies.

The advent of the micro-computer and the rapid developments which have taken place in inexpensive hardware have brought us to the era of a computer on every professional's desk.

The power of these machines is transforming our approach to rock mechanics analysis since it is now possible to perform a large number of sensitivity or probabilistic studies in a fraction of the time which was required for a single analysis a few years ago.

Given the wide scope of numerical applications available today, it has become essential to fully understand the varying strengths and limitations inherent in each of the different methodologies.

For example, limit equilibrium methods still remain the most commonly adopted solution method in rock slope engineering, even though most failures involve complex internal deformation and fracturing which bears little resemblance to the 2-D rigid block assumptions required by most limit equilibrium back-analyses.

Initiation or trigger mechanisms may involve sliding movements which can be analysed as a limit equilibrium problem, but this is followed by or preceded by creep, progressive deformation, and extensive internal disruption of the slope mass. The factors initiating eventual sliding may be complex and not easily allowed for in simple static analysis.

Notwithstanding the above comments, limit equilibrium analyses may be highly relevant to simple block failure along discontinuities. It is the intent of this research thesis to demonstrate that limit equilibrium techniques should be used in conjunction with numerical modelling to maximize the advantages of both.

The argument for the use of all relevant available slope analysis techniques in a design or back-analysis is crystallized by the observation of Chen, “In the early days, slope failure was always written off as an act of God. Today, attorneys can always find someone to blame and someone to pay for the damage – especially when the damage involves loss of life or property”.

The design of a slope using a limit equilibrium analysis alone may be completely inadequate if the slope fails by complex mechanisms (e.g. progressive creep, internal deformation and brittle fracture, liquefaction of weaker soil layers, etc).

Furthermore, within slope engineering design and analysis, increased use is being made of hazard appraisal and risk assessment concepts. A risk assessment must address both the consequence of slope failure and the hazard or probability of failure; both require an understanding of the failure mechanism in order that the spatial and temporal probabilities can be addressed.

2.20 Conventional Methods of Rock Slope Analysis

Table 2.20.1 provides a summary of the techniques that are routinely applied in conventional slope analyses together with their inherent advantages and limitations. As such, the first step in any rock slope stability analysis must be a detailed evaluation of the lithology and rock mass structure, as discussed previously.

From this follows the necessity to determine if the orientation of the existing discontinuity sets could lead to block instability. This assessment may be carried out by means of stereographic techniques and kinematic analysis.

For example, the Rocscience program DIPS allows for the visualisation and determination of the kinematic feasibility of rock slopes using friction cones, daylight and toppling envelopes, in addition to graphical and statistical analysis of the discontinuity properties. It is essential that such approaches recognise potential sliding failures involving single discontinuities or discontinuity intersections.

They do not cater for failure involving multiple joints/joint sets or internal deformation and fracture. Discontinuity data and joint set intersections defined in DIPS, however, can be imported into companion limit equilibrium codes (e.g. SWEDGE (2)) to assess the factor of safety against sliding (Figure 2.20.1).

These programs often incorporate probabilistic tools, in which variations in joint set properties and added support measures can be assessed for their influence on the factor of safety.

All limiting equilibrium techniques share a common approach based on a comparison of resisting forces/moments mobilised and the disturbing forces/moments. Methods vary, however, in the assumptions adopted in order to achieve a determinate solution.

Graphical analysis using stereonet techniques can also be carried out using block theory techniques to assess critical key-blocks. The stability of such key-blocks can then be assessed using limit equilibrium methods such as in the SAFEX program (3) and KBSLOPE.

Analysis method	Critical input parameters	Advantages	Limitations
Stereographic and Kinematic	Critical slope and discontinuity geometry; representative shear strength characteristics.	Relatively simple to use and give an initial indication of failure potential. Some methods allow identification and analysis of critical keyblocks. Links are possible with other analysis methods. Can be combined with statistical techniques to indicate probability of failure and associated volumes.	Only really suitable for preliminary design or design of non-critical slopes. Need to determine critical discontinuities that requires engineering judgement. Must be used with representative discontinuity/joint shear strength data. Primarily evaluates critical orientations, neglecting other important joint properties.
Limit Equilibrium	Representative geometry and material characteristics; soil or rock mass shear strength parameters (cohesion and friction); discontinuity shear strength characteristics; groundwater conditions; reinforcement characteristics and external support data.	Wide variety of software available for different failure modes (planar, wedge, toppling, etc.). Mostly deterministic but increased use of probabilistic analysis. Can analyse factor of safety sensitivity to changes in slope geometry and material behaviour. Capable of modelling 2-D and 3-D slopes with multiple materials, reinforcement and groundwater profiles.	Factor of safety calculations give no indication of instability mechanisms. Numerous techniques available all with varying assumptions. Strains and intact failure not allowed for. Do not consider *in situ* stress state. Probabilistic analysis requires well-defined input data to allow meaningful evaluation. Simple probabilistic analyses may not allow for sample/data covariance.
Rockfall Simulation	Representative slope geometry; rock block sizes and shapes; coefficient of restitution.	Practical tool for siting structures. Can utilise probabilistic analysis. 2-D and 3-D codes available	Limited experience in use relative to empirical design charts.

Table 2.20.1 – Summary of Industry Reorganised Rock Mechanics Software

Considerable advances in commercially available limit equilibrium computer codes have taken place in recent years. These include:

- Integration of 2-D limit equilibrium codes with finite-element groundwater flow and stress analyses (e.g. GEO-SLOPE's SIGMA/W, SEEP/W and SLOPE/W.
- Development of 3-D limit equilibrium methods (e.g. CLARA (7); 3D-SLOPE.
- Development of probabilistic limit equilibrium techniques.
- Ability to allow for varied support and reinforcement.
- Incorporation of unsaturated soil shear strength criteria.
- Greatly improved visualisation, and pre-and post-processing graphics.

These codes are extremely relevant in the analysis of soil slopes and highly altered

rock slopes, where sliding takes place on discrete well-defined surfaces. Figure 2.20.2 illustrates the use of the 2-D limit equilibrium program SLOPE/W in the back-analysis of a failure in a kaolinised granite slope.

Where it is necessary to include the stress state within the rock mass and the influence of complex deformation and brittle fracture, numerical modelling techniques should be used (e.g. Figure 2.20.2).

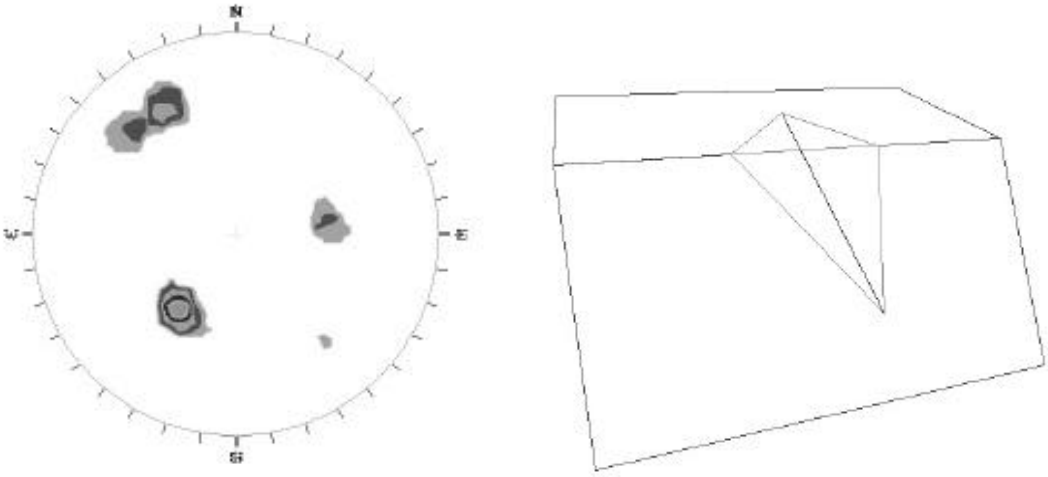


Figure 2.20.1 - SWEDGE analysis (RIGHT) based on DIPS stereonet input (LEFT)

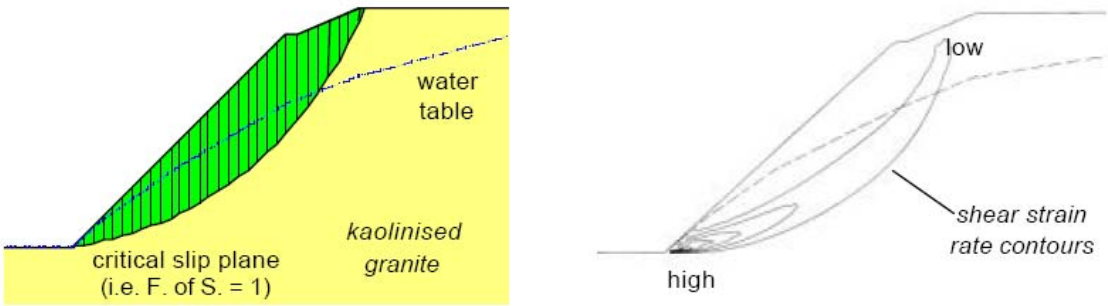


Figure 2.20.2 - Analysis of China clay slope using limit equilibrium to find the critical slip plane

Note:(LEFT) and finite-difference to model shear strain development (RIGHT).

Rockfall simulators, another conventional form of analysis, include tools used to assess hazards of individual falling blocks. Programs such as ROCFALL analyse the

trajectory of falling blocks based on changes in velocity as rock blocks roll and bounce over a given slope geometry.

Other factors solved for include:

- Block velocity,
- Bounce height and
- Endpoint distance.

which can be analysed statistically over a repeated number of simulations to aid in a risk assessment. Rockfall simulators can also assist in determining remedial measures by calculating the effectiveness and kinetic energy of impact on barriers.

Similar developments that deal with failed rock blocks and rapid slides include Hungr's DAN code, which proposes a dynamic analysis tool suited for the prediction of flow and runout behaviour.

2.21 Advanced Numerical Methods of Slope Analysis

Many rock slope stability problems involve complexities relating to geometry, material anisotropy, non-linear behaviour, in situ stresses and the presence of several coupled processes (e.g. pore pressures, seismic loading, etc).

Advances in computing power and the availability of relatively inexpensive commercial numerical modelling codes means that the simulation of potential rock slope failure mechanisms could, and in many cases should, form a standard component of a rock slope investigation.

Numerical methods of analysis used for rock slope stability may be conveniently divided into three approaches: continuum, discontinuum and hybrid modelling. Table 2.21.1 provides a summary of existing numerical techniques.

Analysis method	Critical input parameters	Advantages	Limitations
Continuum Modelling (e.g. finite-element, finite-difference)	Representative slope geometry; constitutive criteria (e.g. elastic, elasto-plastic, creep etc.); groundwater characteristics; shear strength of surfaces; *in situ* stress state.	Allows for material deformation and failure. Can model complex behaviour and mechanisms. Capability of 3-D modelling. Can model effects of groundwater and pore pressures. Able to assess effects of parameter variations on instability. Recent advances in computing hardware allow complex models to be solved on PC's with reasonable run times. Can incorporate creep deformation. Can incorporate dynamic analysis.	Users must be well trained, experienced and observe good modelling practice. Need to be aware of model/software limitations (e.g. boundary effects, mesh aspect ratios, symmetry, hardware memory restrictions). Availability of input data generally poor. Required input parameters not routinely measured. Inability to model effects of highly jointed rock. Can be difficult to perform sensitivity analysis due to run time constraints.
Discontinuum Modelling (e.g. distinct-element, discrete-element)	Representative slope and discontinuity geometry; intact constitutive criteria; discontinuity stiffness and shear strength; groundwater characteristics; *in situ* stress state.	Allows for block deformation and movement of blocks relative to each other. Can model complex behaviour and mechanisms (combined material and discontinuity behaviour coupled with hydro-mechanical and dynamic analysis). Able to assess effects of parameter variations on instability.	As above, experienced user required to observe good modelling practice. General limitations similar to those listed above. Need to be aware of scale effects. Need to simulate representative discontinuity geometry (spacing, persistence, etc.). Limited data on joint properties available (e.g. jk_n, jk_s).
Hybrid/Coupled Modelling	Combination of input parameters listed above for stand-alone models.	Coupled finite-element/distinct-element models able to simulate intact fracture propagation and fragmentation of jointed and bedded media.	Complex problems require high memory capacity. Comparatively little practical experience in use. Requires ongoing calibration and constraints.

Table 2.21.1 Numerical methods of analysis

Continuum Modelling

Continuum modelling is best suited for the analysis of slopes that are comprised of massive, intact rock, weak rocks, and soil-like or heavily fractured rock masses. Most continuum codes incorporate a facility for including discrete fractures such as faults and bedding planes but are inappropriate for the analysis of blocky mediums.

The continuum approaches used in rock slope stability includes the finite-difference and finite-element methods. The salient advantages and limitations are discussed by Hoek et al, and both have found widespread use in rock slope analysis.

In recent years the vast majority of published continuum rock slope analyses have used the 2-D finite-difference code, FLAC. This code allows a wide choice of

constitutive models to characterise the rock mass and incorporates time dependent behaviour, coupled hydro-mechanical and dynamic modelling.

An example of the use of FLAC in the modelling of buckling type failures in a surface coal mine slope is shown in Figure 2.21.1 Two-dimensional continuum codes assume plane strain conditions, which are frequently not valid in inhomogeneous rock slopes with varying structure, lithology and topography.

The recent advent of 3-D continuum codes such as FLAC3D and VISAGE enables the engineer to undertake 3-D analyses of rock slopes on a desktop computer. An example of a FLAC3D analysis of a china clay slope, which incorporated distinct zones of alteration along strike, is shown in Figure 2.21.1.

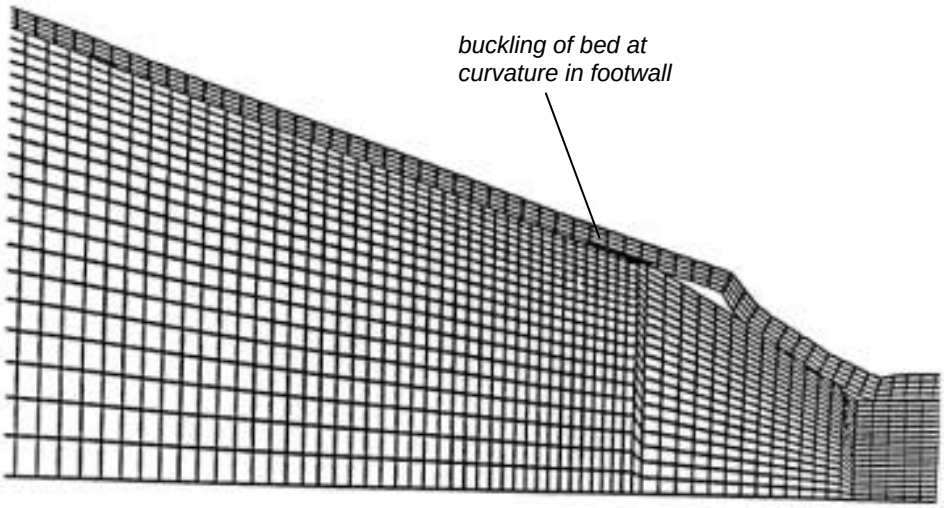


Figure 2.21.1 FLAC model of buckling failure in a surface coal mine slope

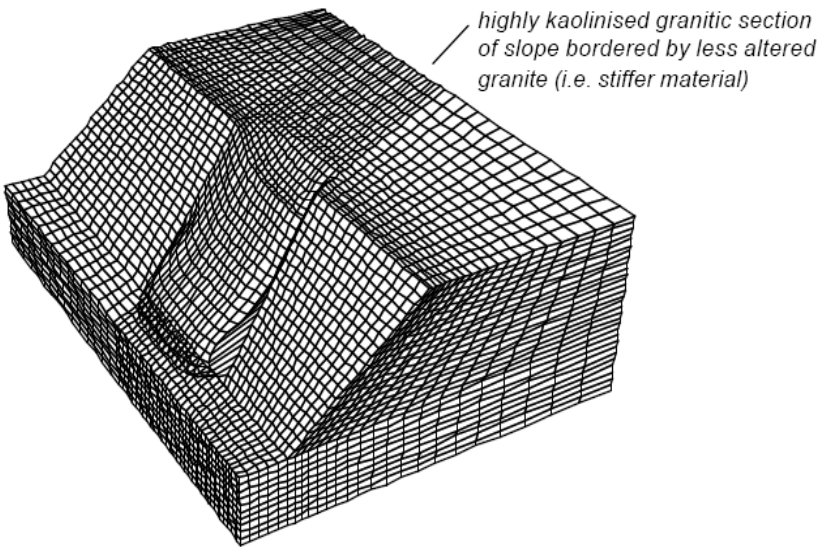


Figure 2.21.2 - FLAC3D model of china clay slope

Although 2-D and 3-D continuum codes are extremely useful in characterising rock slope failure mechanisms it is important to verify whether they are representative of the rock mass under consideration.

Where a rock slope comprises multiple joint sets, which control the mechanism of failure, then a discontinuum modelling approach may be considered more appropriate.

Discontinuum Modelling

Discontinuum methods treat the rock slope as a discontinuous rock mass by considering it as an assemblage of rigid or deformable blocks. The analysis includes sliding along and opening/closure of rock discontinuities controlled principally by the joint normal and joint shear stiffness.

Discontinuum modelling constitutes the most commonly applied numerical approach to rock slope analysis, the most popular method being the distinct-element method. Distinct-element codes such as UDEC use a force-displacement law specifying

interaction between the deformable joint bounded blocks and Newton's second law of motion, providing displacements induced within the rock slope.

UDEC is particularly well suited to problems involving jointed media and has been used extensively in the investigation of both landslides and surface mine slopes. The influence of external factors such as underground mining, earthquakes and groundwater pressure on block sliding and deformation can also be simulated.

Figure 2.21.3 shows an analysis of the Frank Slide, a major rockslide that occurred in Alberta, Canada. This modelling investigation is described in detail by Benko and Stead and illustrates the possible role of underground coal mining at the foot of the mountain slope on the initiation of the rockslide.

Figure 2.21.4 illustrates the use UDEC in the modelling of a major toppling instability at the Luscar Mine, Alberta, Canada. This analysis was able to simulate the progressive development of a basal flexure surface as mining proceeded with depth from the surface. By undertaking a program of numerical analyses on both observed stable and unstable slopes, the modelling was able to provide valuable information for future mine planning.

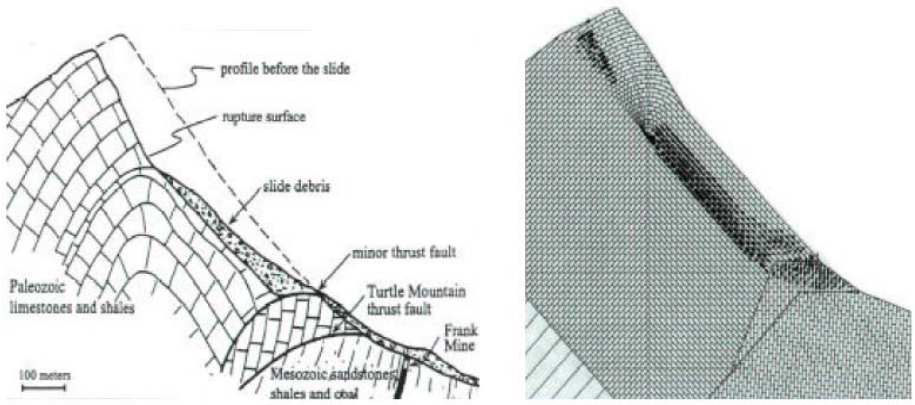


Figure 2.21.3. Schematic section of Frank ***Slide (LEFT) and UDEC model showing shear along bedding and joints (RIGHT)***

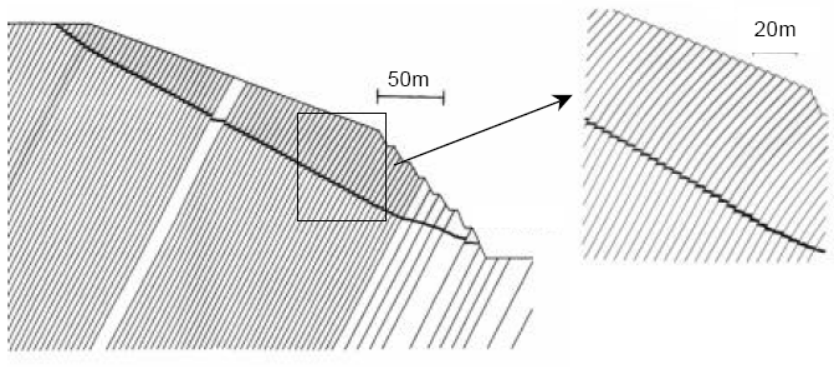


Figure 2.21.4. UDEC model of flexural toppling in a surface coal mine slope

It is important to note that the structural input into the distinct-element analysis is representative. Hencher et al illustrated the importance of bedding spacing on predicted failure mechanism. Stead and Eberhardt showed the importance of discontinuity orientation on failure modes observed in surface coal mine slopes.

It is stressed that tailoring the structure of the model to accommodate the low random access memory of a laptop computer, for example by using unrepresentative discontinuity spacing, may lead to unrepresentative results.

Simulations must always be verified with field observations i.e. calibrated and wherever possible instrumented slope data. This becomes even truer with the development of 3-D discontinuum codes such as 3DEC.

Only when a confident portrayal of the 3-D characteristics of a slope has been obtained, can the results be considered representative. This in turn requires the undertaking of an extensive and in-depth site investigation beforehand, as was discussed in earlier sections.

The discontinuous deformation analysis, DDA, developed by Shi has also been used with considerable success in the modelling of discontinuous rock masses, both in terms of rockslides and rockfalls.

An important recent development in discontinuum codes is the application of distinct-element methodologies and particle flow codes, e.g. PFC2D/3D. This code allows the rock mass to be represented as a series of spherical particles that interact through frictional sliding contacts.

Clusters of particles may be bonded together through specified bond strengths in order to simulate joint bounded blocks. One major advantage of such an approach is that high stresses induced in the rock slope will break the bonds between the particles simulating, in an approximate manner, the intact fracture of the rock.

The use of discontinuum methods in association with continuum methods has been shown by several authors to provide an instructive approach to rock slope analysis. Board et al illustrate the analysis of complex deformation within the 650m high Chuquicamata pit slope, Chile, using a combined approach which utilises both FLAC and UDEC analyses.

Similarly, Benko and Stead used an approach adopting FLAC for the initial investigation of Frank Slide and UDEC for the in-depth analysis. The latter study integrated results from limit equilibrium, continuum and discontinuum analyses using each technique as a tool to provide a step in the overall rock slope analysis.

2.22 Undertaking a Slope Stability Analysis

For typical open pit mines, stability analyses are conducted in two broad parts namely:

- **Batter scale stability analysis** – This analysis is primarily based on kinematics (based on insitu geological structure), i.e. batter slope geometry, including face

angles and catchment berm widths are dependent upon the persistence of structures and their respective orientations to the pit walls. It should be noted that when designing pits in very weak rock masses the influence of geological structure may not have the most pronounced affect on slope geometry, whereby slope geometry would be governed by the strength of the rock mass and effects of pore water pressures (if present).

- **Inter-ramp & Overall scale stability analysis –** This analysis takes into account the batter slope geometry that has been formulated on the basis of kinematics and assesses the interaction of large scale structure and the insitu rock mass, to derive limitations on inter-ramp 'stack heights' and the ideal location and widths of 'geotechnical' berms or haul ramps to effectively decouple the overall slope height.

2.23 Batter Scale Stability Analysis

As mentioned previously the batter scale analysis are essentially kinematic analysis that assess the likelihood that a structure falls within the daylight envelope of a slope i.e. strikes approximately ±30° of the slope Hoek & Bray (1974) and has a trace length that is greater than half the proposed batter height.

This type of analysis is typically conducted on a stereonet, which does not account for the effect of cohesion i.e. it is a 'friction only' analysis, nor does it take into account the trace length of structures. Hence the overall result may provide a pessimistic representation of stability conditions.

The McMahon technique, McMahon (1985) to assess the likelihood of kinematic failure mechanisms is more appropriate, which accommodates the effects of cohesion by converting it into a pseudo friction angle using the following relationship, as shown in Figure 2.23.1:

$$\phi'' = \tan^{-1}(c/\sigma_N + \tan(\phi))$$ - Equation 2.5

Where;

ϕ'' – *Calculated pseudo friction angle*

c **-** *Cohesion*

σ_N – *Normal Stress (Confinement)*

ϕ **-** *Friction Angle*

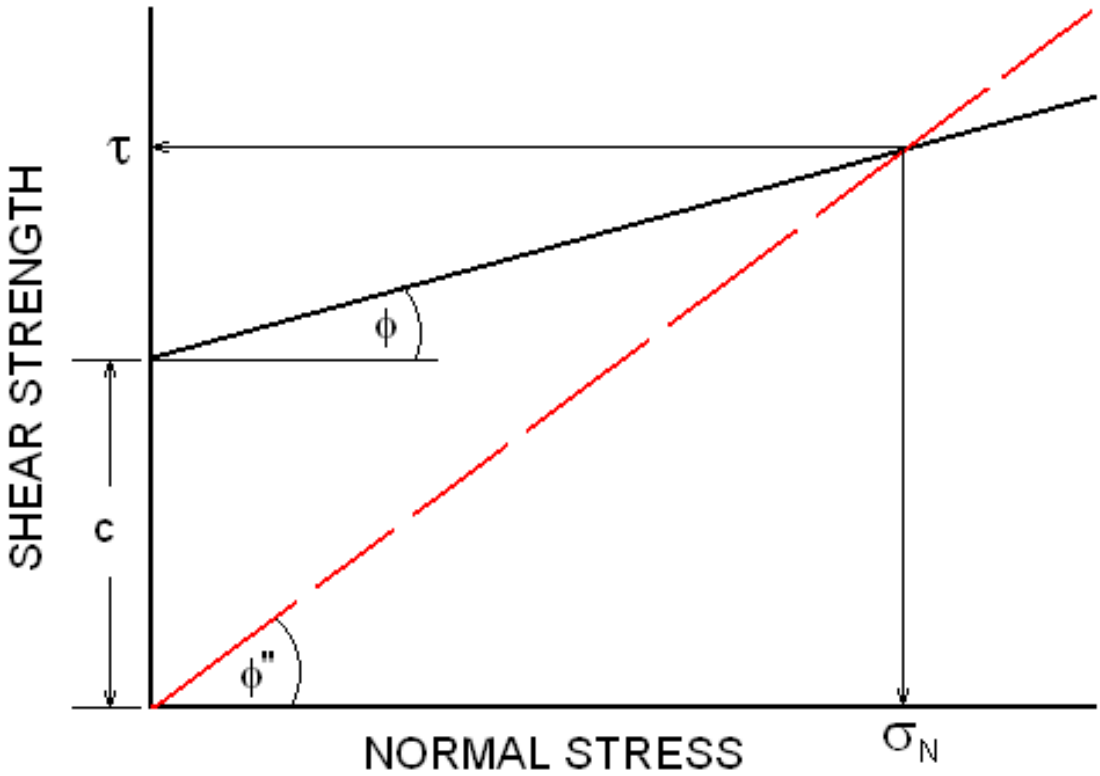


Figure 2.23.1 Calculation of Pseudo Friction Angle from Cohesion, McMahon (1985)

The McMahon technique can be enhanced by the use of a spreadsheet within which all recorded structures can be included, i.e. all the poles as recorded within a stereonet can be used as part of this analysis, rather than using average contoured sets, as one might do within a limit equilibrium software package like Swedge or Rocplane.

A typical output from the McMahon analysis spreadsheet is shown below, the red and blue curves depict the likelihood (probability) of tetrahedral wedge or planar failures respectively at various batter face angles (horizontal axis).

It can be seen from this chart that should one want to achieve a Probability of Failure (PFailure) of less than 20 percent on the batter scale for the given slope height, face angles of 60° or less are required.

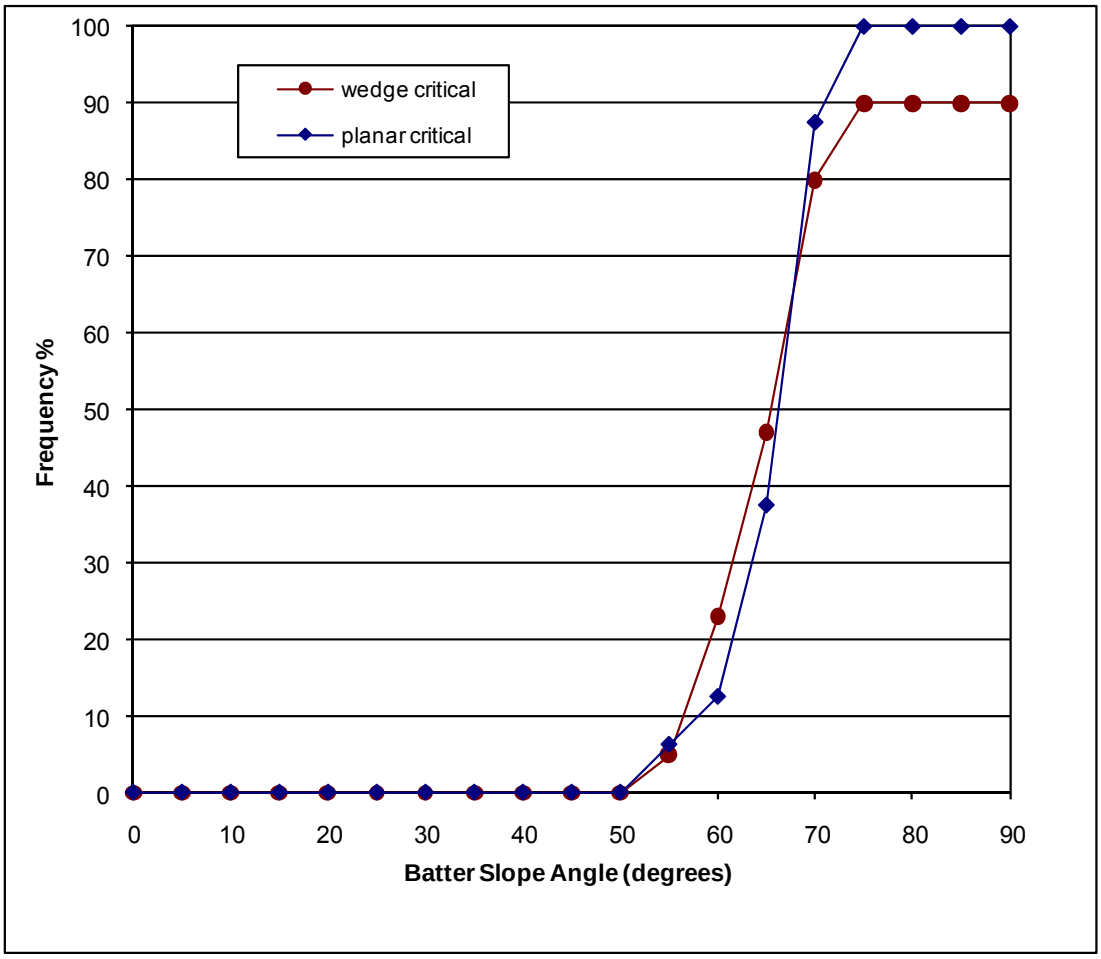


Figure 2.23.2 Output Chart from Kinematic Analysis for the Batter Slope

2.24 Inter-Ramp and Overall Slope Stability Analysis

Overall and inter-ramp scale slope stability analysis can be quite complex as it involves a number of interactions, i.e. materials of different shear strengths (i.e. different lithological units) and their combined performance with large scale geological structure.

Pervious discussed were how the components of material shear strengths i.e. cohesion and friction can be formulated probabilistically, this information can be inputted within a within a software package like Slide to calculate the likely failure geometry and the distribution of Factors of Safety (FOS) and hence probability of failure.

This probability of failure (PFailure) can be used to determine the reliability of a mine slope whereby: ***Reliability (R) = 1 – PFailure,*** R can be interpreted as the likelihood that a particular slope would perform adequately under insitu conditions i.e. accounting for variabilities within material (lithology) and geological structure, for a given period of time, Harr (1987).

The figures below shows the output (FOS distribution) and cumulative distribution of FOS from a probabilistic analysis conducted on Slide. It can be seen that given the variabilities associated with the input parameters for this analysis, the mean FOS is approximately 1.25 and the PFailure i.e. the probability that the calculated FOS is less than 1.0 is 7 percent. This implies that the slope has a reliability of 93 percent. The reliability (a measure of the 'risk' associated with the design) of the slope design can be used to potentially optimise slope angles.

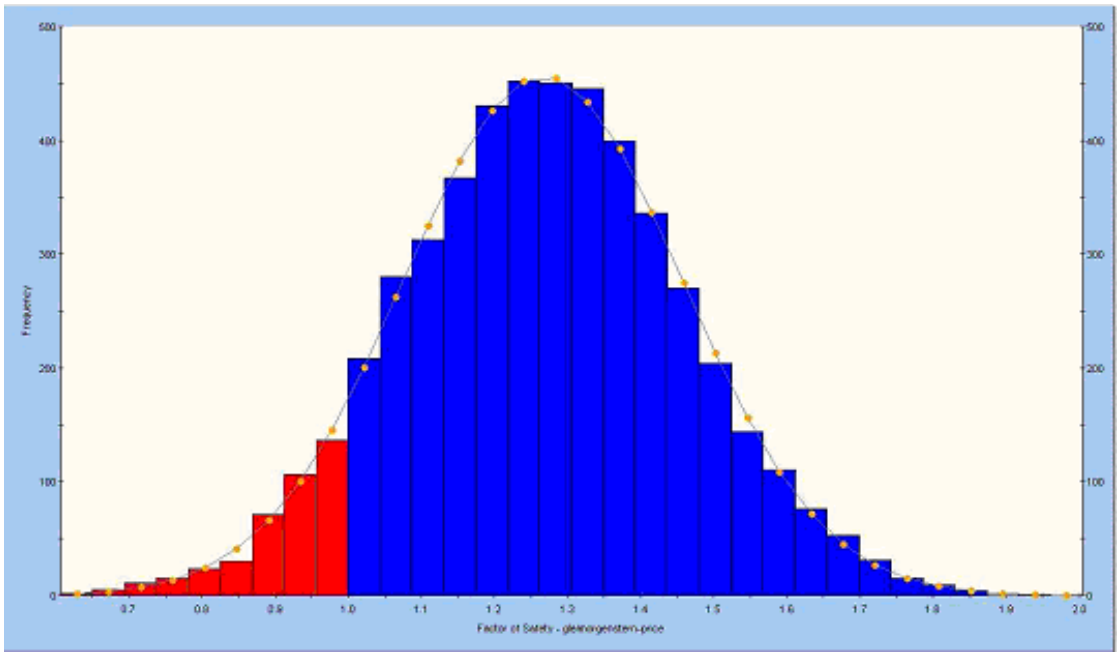


Figure 2.24.1a Distribution of FOS's from Slide

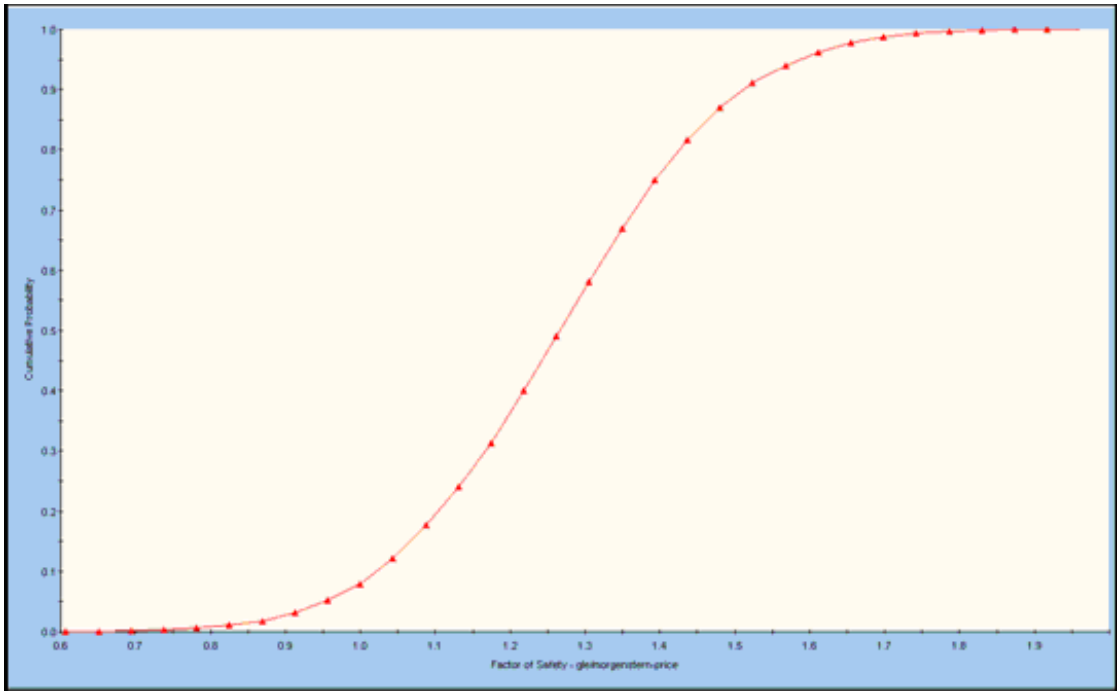


Figure 2.24.1b Cumulative Probability Function from Slide

2.25 Mine (Pit) Optimisation and Slope Design Reliability

The pit optimisation process can be quite simply defined as the payoff that could potentially be received for accepting a particular level of risk. If slope designs were done deterministically i.e. where only a single FOS was calculated and the variability and uncertainty associated with the input parameters was not known or quantified, the risk associated with a slope design cannot be determined.

Risks Associated with Slope Failure

The risks associated with taking a more 'aggressive; design approaches are as follows as identified by Lilly (2000):

- **Clean up costs** – The cost of mobilising and operating equipment, which would otherwise be used for mine production to clean up failure masses on the pit floor, so that regular mining activities can resume.
- **Slope re-formation** – Depending on the type of failure mechanisms, for example rotational failure, steep back scarps may be left behind, that require battering back (flattening) so as not to initiate further failures, this is sometimes carried out in parallel with regular pit floor cleanup activities.
- **Haul road repair and re-access** – Should a particular slope failure, take place onto a haul ramp or compromise its integrity by occurring below it and undercutting or occur above it, access to the pit will be lost until a alternate access can be restabilised, if the ramp is the sole access to the pit. This would result in significant production losses albeit temporarily.
- **Unrecoverable ore** – In an extreme case there is the potential for a slope failure to bury an orebody or at least a critical blend required for processing purposes, and the consequences of this failure becomes significant as production ceases entirely until the area is remediated and the ore recovered.

- **Damage to equipment, personnel and infrastructure** – Where key pieces of infrastructure (decline portals, process facilities, public roads and highways etc), equipment or personnel are damaged or injured as a result of a slope failure, the consequences of the failure becomes quite large.

Therefore it can be said that the total 'consequence' of a slope failure would be a combination of the costs of all of the above items. This information can be presented as follows as depicted within Figure 2.25.1, where the total cost of the failure (encompassing all of the components) can be expressed in relation to the slope angle.

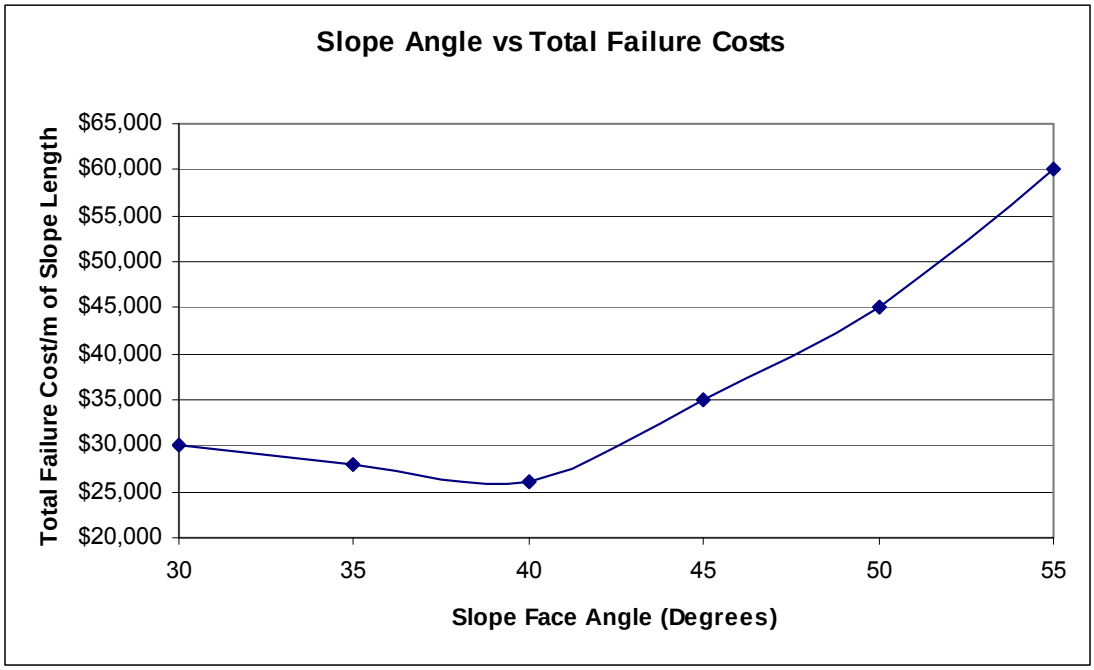


Figure 2.25.1 Total Cost Analysis

2.26 Optimising a Pit Slope

The fundamental item to bear in mind when considering optimising a mine slope within a particular geotechnical domain is the consequence of the failure, i.e. the sum of the costs of the items identified above, in relation to the failure volume.

It should be noted that for 'long life' pits this process may have to be carried out over a range of pit depths as the time value of money which strongly influences the projects net present value (NPV) may change over time.

Within a particular slope domain the anticipated failure volume (Vf) can be estimated based on the calculated failure geometry from modelling and the associated slope face angles. However the actual failure will depend on the uncertainty of the model i.e. the probability of failure, hence:

VExpected Failure = PFailure x Vfailure.

The reliability of the slope design and anticipated consequence (cost) of failure can be plotted on the one chart to assist the mining engineer to select an optimum slope angle to form the slope. If the mining operation is prepared to 'accept' a certain degree of failure which can be included within the mining costs, then a lower reliability of slope design can be accepted, i.e. a steeper angle.

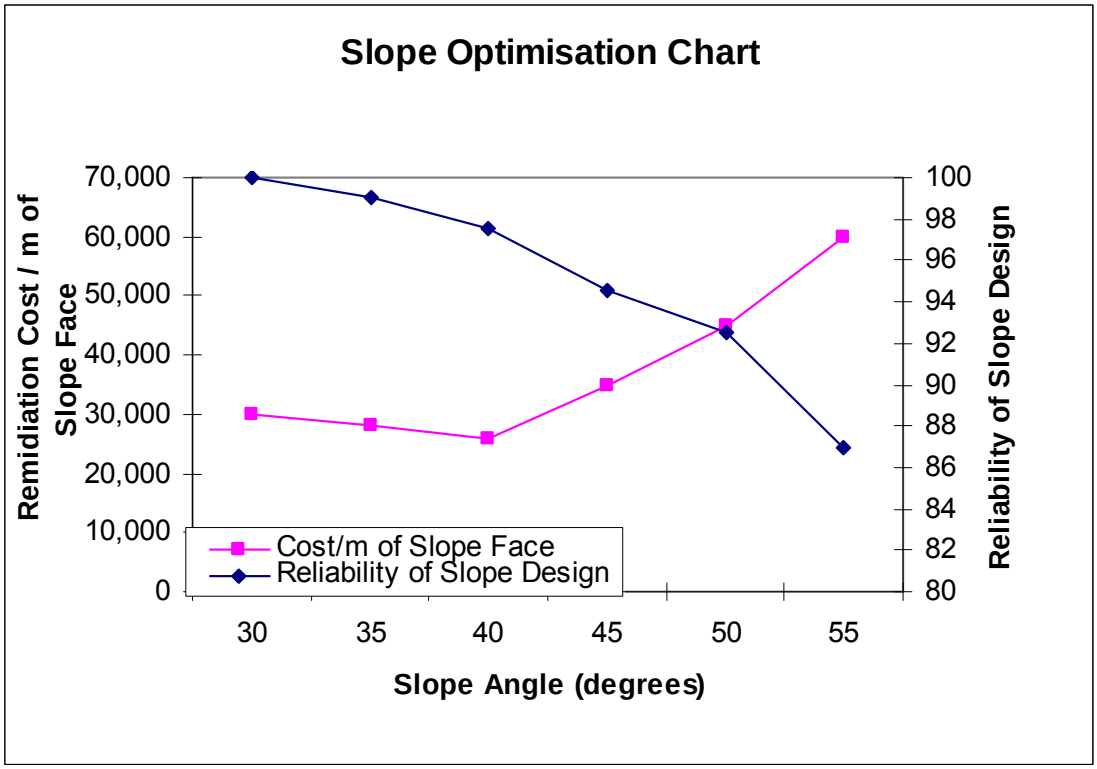


Figure 2.29 Slope Optimisation Chart

2.27 Overall Benefits of Probabilistic Slope Design

Probabilistic slope design has a number of benefits in comparison to deterministic slope design as it addresses uncertainty of a particular design and gives an indication of the reliability of the calculated FOS.

It can also be used to optimise slope angles based on a quantified level of risk (probability of failure / expected volume of failure). However the undertaking of a probabilistic design requires data to be collected in a manner which allows this, i.e. in such a manner to assess the variability of each lithological unit, using say the RMR_{89} system or similar, essentially a system to quantify the variability.

The potential benefits that may result from undertaking a probabilistic design can be listed as follows:

- Once the variability of the design has been assessed, expected volumes of failure can be calculated, this information can be fed into 'fed' into a pit

optimisation programme i.e. Whittle to provide a pit shell(s) based on the optimum slope angle given a set of variabilities be they with the geotechnical or resource model.

- This would facilitate better scheduling requirements, i.e. as the mine scheduling can now incorporate expected volumes of failure to make an estimate on clean up times and equipment requirements.
- This uncertainty of the geotechnical model can be inputted into spreadsheets that calculate Net Present Value (NPV) and Internal Rate of Return (IRR) or any other method for tracking performance.

Safety during construction and long term stability are factors that have to be considered by the designers of excavations in rock. It is not unusual for these requirements to lead to a need for the installation of some form of rock reinforcement or support.

Fortunately, practical developments in this field have been significant during the past 25 years and today a wide choice of reinforcement systems and slope face lining techniques are available. In particular, the development of shotcrete has made a major contribution to modern slope construction.

There has been considerable confusion in the use of the terms “reinforcement” and “support” in rock engineering and it is important to understand the different roles of these two important systems.

Rock reinforcement, as the name implies, is used to improve the strength and/or deformational behaviour of a rock mass in much the same way that steel bars are used to improve the performance of reinforced concrete.

The reinforcement generally consists of bolts or cables that are placed in the rock mass in such a way that they provide confinement or restraint to counteract loosening and movement of the rock blocks.

They may or may not be tensioned, depending upon the sequence of installation, and they may or may not be grouted, depending upon whether they are temporary or permanent.

In general, rock reinforcement is only fully effective in reasonably frictional rock masses of moderate to high strength. Such rock masses permit effective anchoring of the reinforcement and they also develop the interlocking required to benefit from the confinement provided by the reinforcement.

In reinforced rock masses, mesh and/or shotcrete play an important role in bridging the gap between adjacent bolt or anchor heads and in preventing progressive ravelling of small pieces of rock that are not confined by the reinforcement.

For weak to very weak rock masses that are more cohesive than frictional, reinforcement is less effective and, in the case of extremely weak materials i.e. flyash, may not work at all. In these cases it is more appropriate to use support rather than reinforcement.

This support, which generally consists of steel sets and shotcrete or concrete linings in different combinations, must act as a load bearing structural shell to be fully effective in failing weak ground.

The primary function of the support is to limit deformation of the rock or soil mass surrounding the slope and the sequence of installation, in relation to the advance of the mine face, is critically important.

The capacity of the structural shell must be calculated on the basis of the bending moments and axial thrusts that are generated in the support elements and connections.

In the case of slopes in very weak, highly stressed ground, where multiple faces and benches are being worked, temporary internal support shells may be required in order to prevent collapse of the temporary excavation boundaries.

The development of shotcrete has been extremely important in weak ground construction since it permits the rapid installation of a temporary or permanent load bearing lining with embedded reinforcement as required.

The use of long untensioned grouted cables in underground hard rock mining Clifford, (1974), Fuller, (1983), Hunt and Askew, (1977), Brady and Brown, (1985) has been a particularly important innovation which has resulted in significant improvements in safety and mining costs in massive ore bodies and in open pit slopes.

The lessons learned from these mining systems have been applied with considerable success in civil engineering and the use of untensioned dowels, installed as close as possible to the advancing face, has many advantages in high speed tunnel construction and mine slope face exposure.

The use of untensioned grouted cables or reinforcing bars has also proved to be a very effective and economical technique in rock slope stabilisation. This reinforcement is installed progressively as the slope is benched downward and it is very effective in knitting the rock mass together and preventing the initiation of ravelling.

The design of both rock reinforcement and support have benefited greatly from the evolution of personal computers and the development of very powerful and user-friendly software.

Whereas, in the past, these designs were based on empirical rules or classification schemes derived from experience, it is now possible to study a wide range of excavation geometries, excavation sequences, rock mass properties and reinforcement or support options by means of numerical models.

This does not imply that every metre of every excavation has to be subjected to such analyses but it does mean that, once a reliable geological model has been established, a few reinforcement or support systems can be chosen and optimised for the typical conditions anticipated.

2.28 Operational Considerations in Rock Excavation

As mentioned earlier in this thesis, the strength of jointed rock masses is dependent upon the interlocking between individual rock pieces. This interlocking is easily destroyed and careless blasting during excavation is one of the most common causes of slope excavation instability. The following quotation is taken from a paper by Holmberg and Persson (1980).

"*The innocent rock mass is often blamed for insufficient stability that is actually the result of rough and careless blasting. Where no precautions have been taken to avoid blasting damage, no knowledge of the real stability of the undisturbed rock can be gained from looking at the remaining rock wall. What one sees are the sad remains of what could have been a perfectly safe and stable rock face.*"

Techniques for controlling blast damage in rock are well-known Svanholm et al, (1977), Langefors and Kihlstrom, (1963), Hagan, (1980) but it is sometimes difficult to persuade owners and contractors within mining operations that the application of these techniques is worthwhile.

Experience in projects in which carefully controlled blasting has been used generally shows that the amount of reinforcement can be reduced significantly and that the

overall cost of excavation and support is lower than in the case of poorly blasted excavations, Hoek (1982). Examples of poor and good quality blasting in slopes are illustrated in Figures 2.28.1a and 2.28.1b.

Figure 2.28.1a – Example of a Well Formed Slope Face (Good Blasting)

Figure 2.28.1b – Example of a Poorly Formed Slope Face (Poor Blasting)

Machine excavation is a technique which causes very little disturbance to the rock surrounding a slope excavation. A wide range of surface mining machines have been developed over the past 25 years and these machines are now capable of working in almost all rock types Robbins, (1976), McFeat-Smith, (1982). Further development of these machines can be expected and it is probable that machine excavation will play a much more important role in future slope formation than it does today.

3 INTRODUCTION TO CASE STUDIES

This section of the thesis presents detailed geotechnical assessments on four lithological units (Case Studies) undertaken as part of this research, which illustrate the benefits of utilising the Rock Mass Variability Index (RMVI) approach to selecting material properties and undertaking slope stability assessments.

The overall aim of the case studies is to demonstrate the application of the RMVI process mentioned within Section 1 of this thesis. The case studies are presented in the following comportment:

Case Study A

The information presented here consists of data gathered from an investigative drilling programme conducted on a moderately strong to strong foliated sedimentary rock formation, which is inter-bedded with weaker material, typically low friction clay infilling. The aim of the case study is to demonstrate that the application of the RMVI approach has the upside of optimising slope design parameters which will lead to a reduction in waste movement and hence an increase in mine operating revenue.

Case Study B

The information presented here consists of data gathered from an investigative drilling programme conducted on a weak to very weak bedded sedimentary shale material. The aim of this case study is to depict the importance of understanding the uncertainty associated with the respective material parameters, using the RMVI approach,, so as not to produce a somewhat aggressive slope design, which can is endemic to low strength materials. These designs typically are not achievable and result in a higher likelihood of unplanned slope failure and hence cost.

Case Study C

The information presented here consists of data gathered from an investigative drilling programme conducted on a moderately strong to strong foliated metamorphic rock formation, which is inter-bedded with strong high friction material, i.e. hematite infilling. The aim of the case study is to demonstrate that the application of the RMVI approach has the upside of optimising slope design parameters which will lead to a reduction in waste movement and hence an increase in mine operating revenue.

Case Study D

The information presented here consists of data gathered from an investigative drilling programme conducted on a moderately strong to strong foliated metamorphic rock formation, which is inter-bedded with weak low friction material, i.e. clay infilling. The aim of the case study is to highlight the importance of assessing defect plane shear strengths and their influence on slope design; as well as to demonstrate the application of the RMVI approach which has the upside of optimising slope design parameters which will lead to a reduction in waste movement and hence an increase in mine operating revenue. Rock Mass Variability Index

The geotechnical models presented as part of the subsequent case studies within this thesis were formulated utilising appropriately weighted rock mass and defect plane strength parameters obtained from both the empirical and analytical avenues.

The methodology used to weight these parameters i.e. obtaining design values from both the analytical and empirical methods for the use in stability modelling has been done using the Rock Mass Variability Index (RMVI) method, which is a technique that has been developed as a result of this research for the determining design values for the respective material parameters requisite for slope stability modelling.

The RMVI method chooses design values from empirical and analytical sources; and adequately quantifies the variability associated with the data sets from both of these sources. The primary reason for combining the information from both these sources is that the empirical methods are typically obtained from sources such as core logging or wall mapping and these methods tend to cover a larger area as opposed to most analytical methods such as laboratory testing which tend to sample discrete points and does not adequately quantify the variability of the respective design input parameters.

The process by which the RMVI is calculated for a set of data is as follows:

$$\textbf{RMVI} = \frac{\{\textbf{Coefficient of Variability (COV)} - \textbf{(Standard Deviation / Mean) x 100}\}}{\textbf{Number of Samples (N)}} \quad \text{- Equation 3.1}$$

The following ranges of RMVI in relation to weighting factors are proposed. These are applied within the case studying in the following sections. The derivation and calibration of these factors is further detailed within Section 10 of this thesis.

RMVI	Reliability Coefficient	Comment
<3	0.99	Extremely Reliable Data Set; Low Insitu Variability & Significant Number of Samples
3 to 7.5	0.85	Very Reliable Data Set; Some Insitu Variability & Significant Number of Samples
7.5 to 20	0.65	Reliable Data Set; Insitu Variability Noted & Significant Number of Samples
20 to 50	0.50	Moderately Unreliable Data Set; Significant Insitu Variability Noted & Inadequate Number of Samples
50 to 100	0.35	Very Unreliable Data Set; Significant Insitu Variability Noted & Inadequate Number of Samples
100 to 150	0.15	Extremely Unreliable Data Set; Significant Insitu Variability Noted & Inadequate Number of Samples
>150	0.05	Inadequate data to determine statistical validity

Table 3.1.1 – RMVI Ranges in Relation to Weighting Factors

The table above provides guidelines as derived from this research for assessing the reliability of a particular set of data. Typically this is not done as part of most rock

mass assessments; i.e. there is a tendency to use design parameters that have been averaged from the analytical or empirical sources.

The negative aspect of formulating design parameters in this manner is that it does not give adequate cognisance to the reliability (confidence) of the respective parameters. The RMVI method of determining weighting factors for the respective design inputs provides a quantitative methodology for determining parameters for rock mass design purposes.

Within the context of an open pit this would involve the following parameters:

1. **Rock mass shear strength** (strength of the intact rock and small scale defects);
2. **Defect plane shear strength** (shear strength mobilised as a result of asperities along a geological structure i.e. bedding or in the case of thick infill material the shear strength of the infill material.

The following sub-sections provide the methodology by which the RMVI principal maybe applied for rock mass design purposes. Sections 4 and 5 present the case studies which involve applying the RMVI principal to formulate design parameters from given sets of empirical and analytical data.

The data sources provided within this case study have been obtained from the following sources:

- **Empirical** - Data collected from diamond core logging.
- **Analytical** - Laboratory testing carried out on samples taken from drill core.

4 CASE STUDY A

4.1 Empirical Data from Core Logging

For the purposes of applying the RMVI methodology information recorded from a core logging campaign consisting of a number of orientated diamond drill holes was utilised. The data from these holes has been presented statistically for individual geotechnical parameters. These parameters consist of:

- **Strength Index**, based on the International Society of Rock Mechanics (ISRM) classifications;
- **Joint Roughness Coefficient** (JRC), after Barton (1974) criteria ;
- **Infill Type**;
- **Infill Width**;
- **Rock Quality Designation** (RQD); and
- **Rock Mass Rating** (RMR_{89}), after Bieniawski (1989).

4.2 Statistical Summary of Empirical Data

The primary geological unit that was assessed as part of Case study A is a moderately strong sedimentary rock unit; which contains a pervasive foliation as the predominant fabric that comprises the overall rock mass.

Geotechnical Parameter	Mean	Mode/ Modal Class	Median	Standard Deviation	Coefficient of Variability (COV)	RMVI
Strength Estimates (MPa) 1055 Samples	38.80	20.00	20.00	34.80	89.7	8.50
JRC 1311 Samples	10.04	16.00	10.00	6.20	59.6	4.55
Infill Width (mm) 1311 Samples	0.60	0.10	0.20	0.90	150	11.44
RQD (%) 559 Samples	36.03	0.00-10.00	27.80	36.20	100.5	17.97
RMR89 559 Samples	41.10	30.00-35.00	37.00	12.60	30.6	5.48

Table 4.2.1– Statistical Summary of Empirical Rock Mass Parameters Lithology A

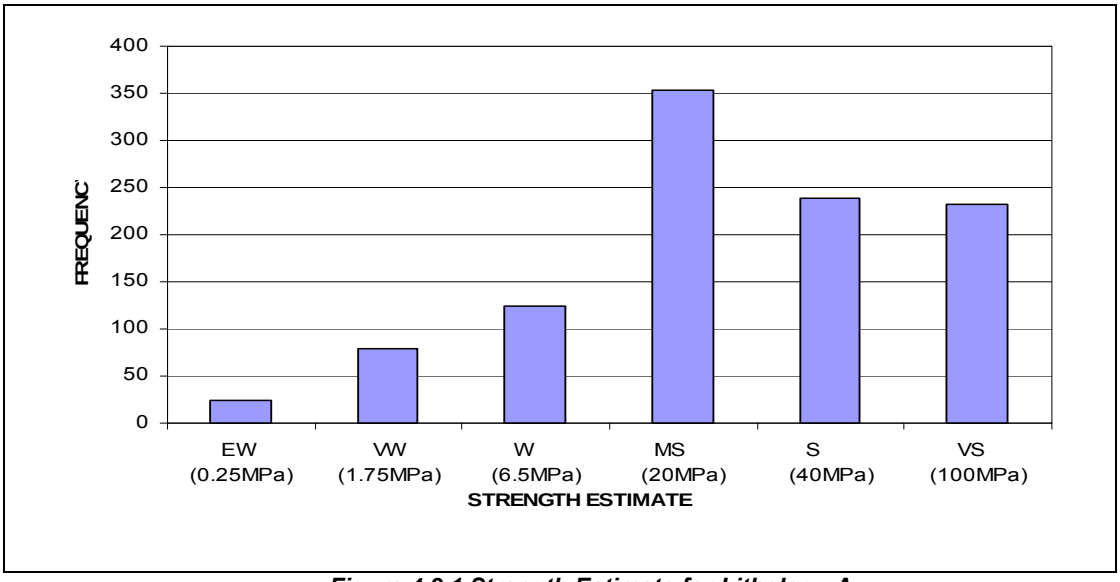


Figure 4.2.1 Strength Estimate for Lithology A

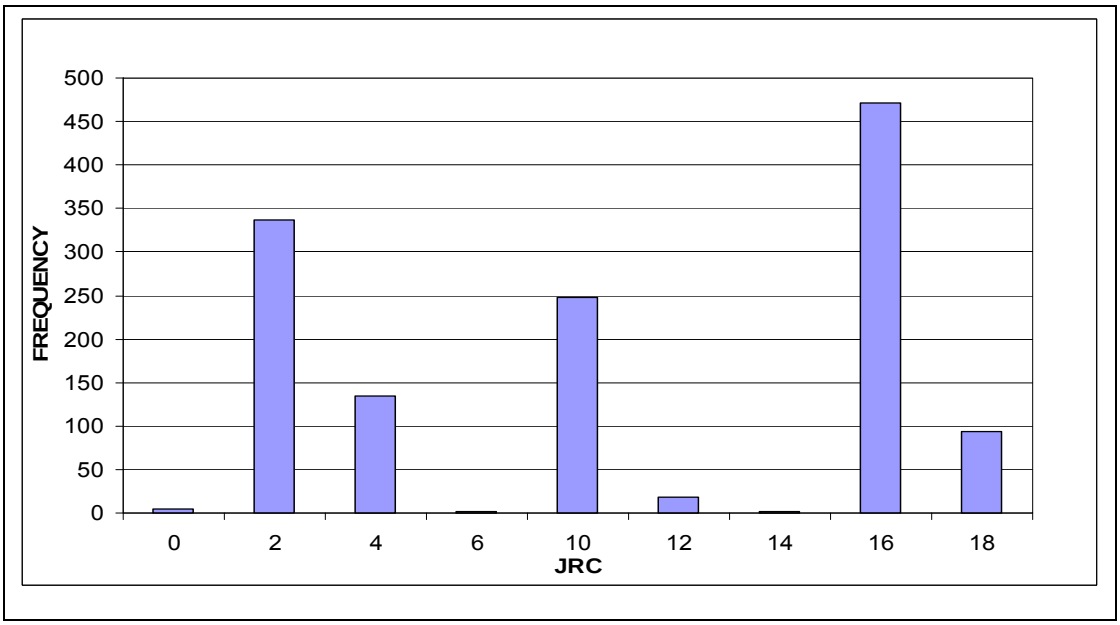


Figure 4.2.2 JRC for Case Lithology A

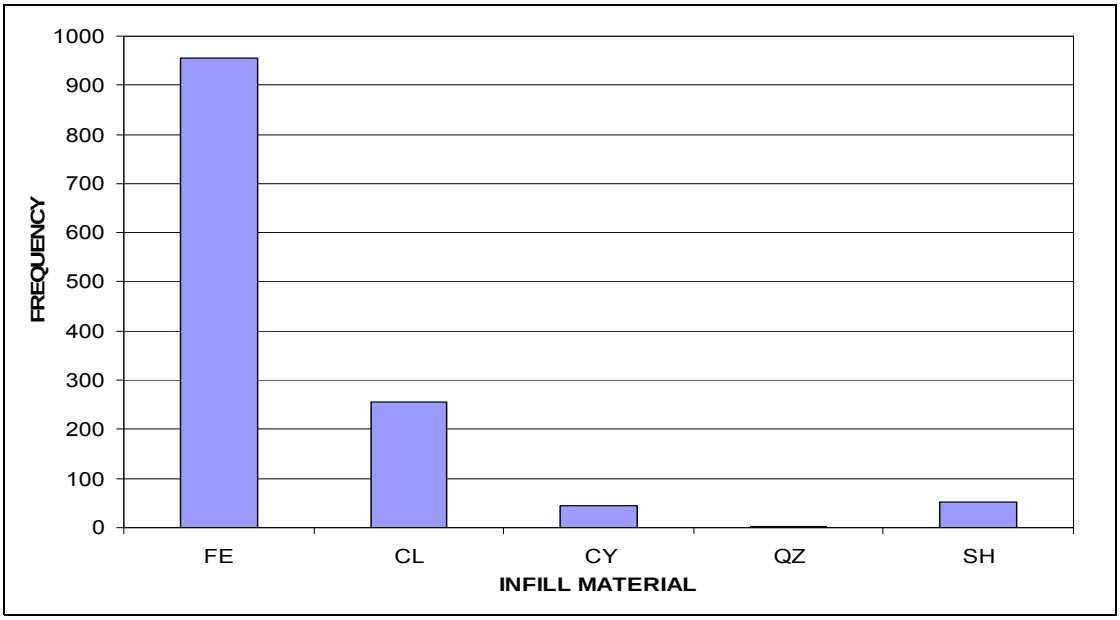


Figure 4.2.3 Infill Type for Case Lithology A

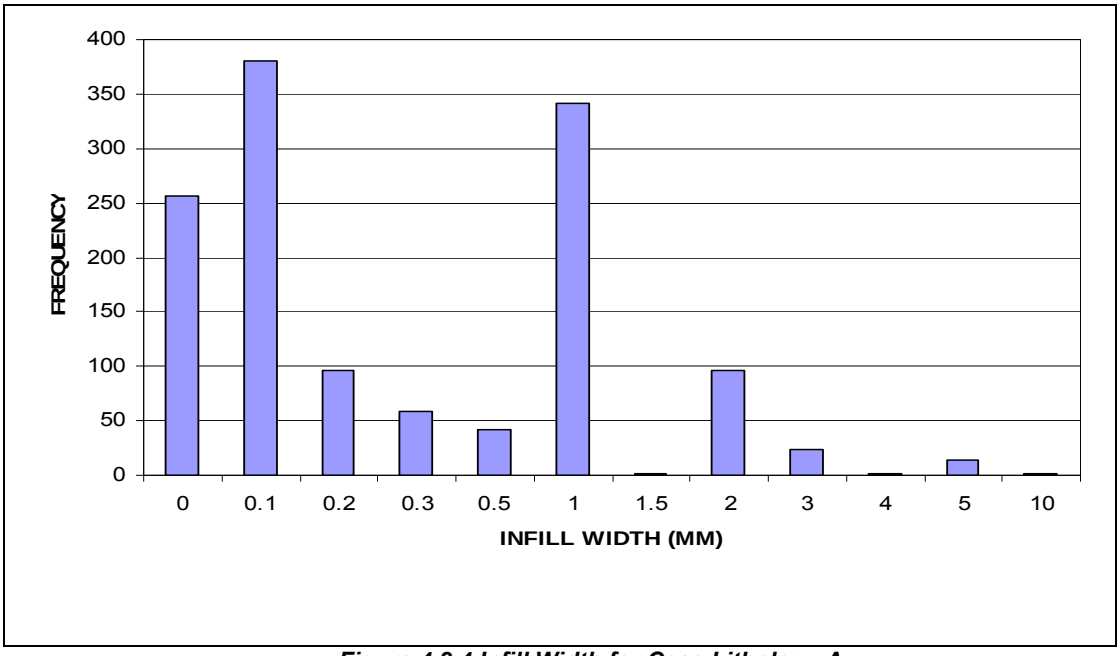


Figure 4.2.4 Infill Width for Case Lithology A

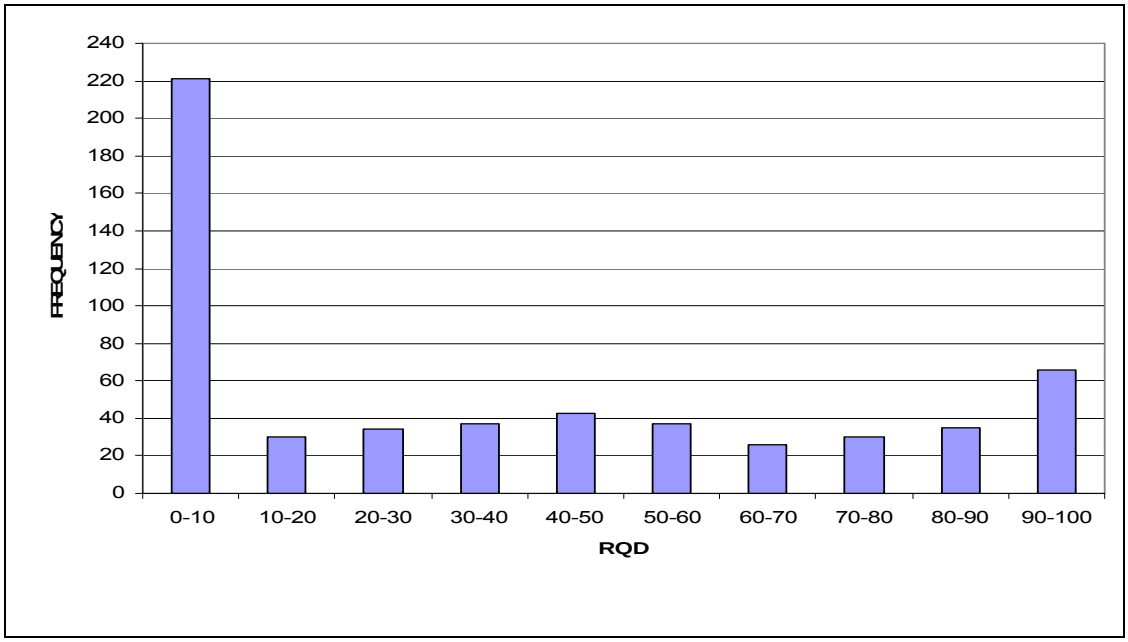


Figure 4.2.5 RQD for Lithology A

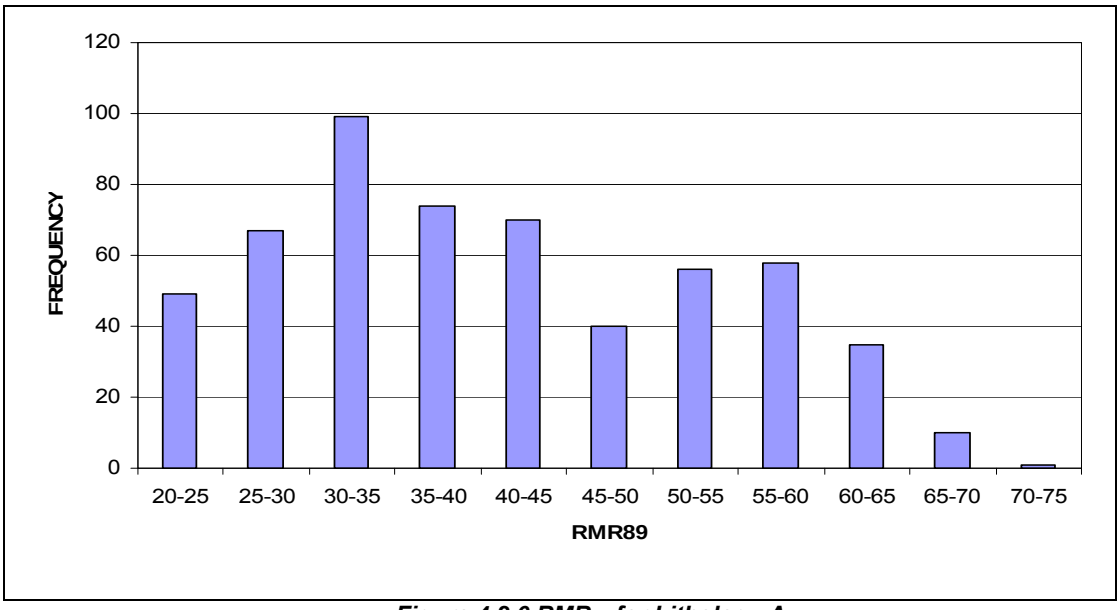


Figure 4.2.6 RMR_{89} for Lithology A

4.3 Statistical Summary of Analytical Data

The RMVI values for each of the three forms of laboratory testing were calculated to determine the statistical integrity of the respective parameters; as presented below.

UCS Data

Statistical Summary	
Mean	85.17
Median	42.00
Standard Deviation	110.36
Coefficient of Variability	129.59
Number of Samples	8.00
RMVI	1619.83

Table 4.3.1 – Statistical Summary of Analytical Data (UCS) from Lithology A

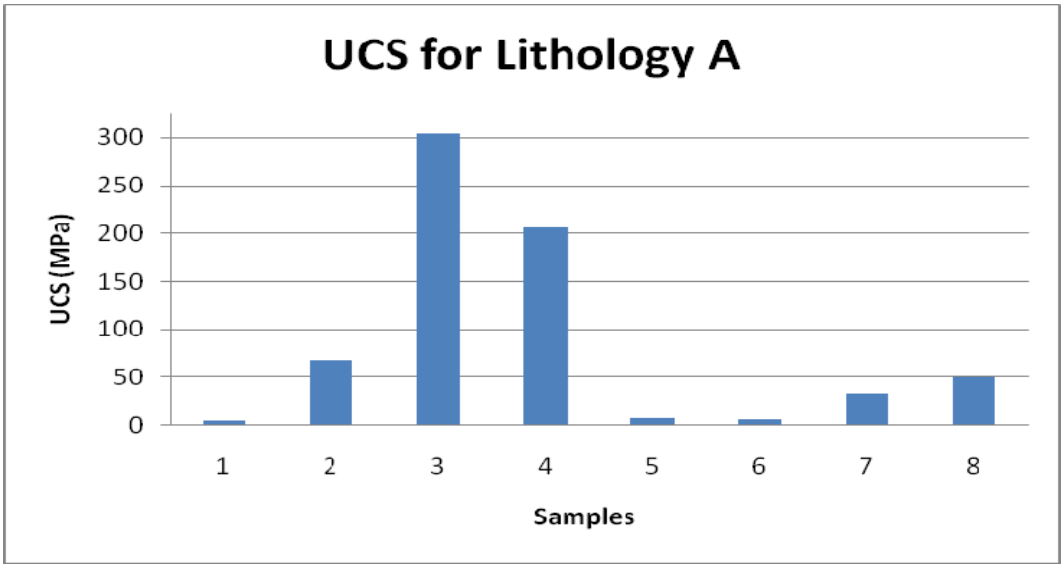


Figure 4.3.1 –UCS Data for Lithology A

Direct Shear Data

Statistical Summary Cohesion (kPa)	
Mean	**187.78**
Median	**110.00**
Standard Deviation	**222.53**
Coefficient of Variability	**118.51**
Number of Samples	**9**
RMVI	**1316.73**

Table 4.3.2– Distribution of Cohesion Values from Direct Shear Testing

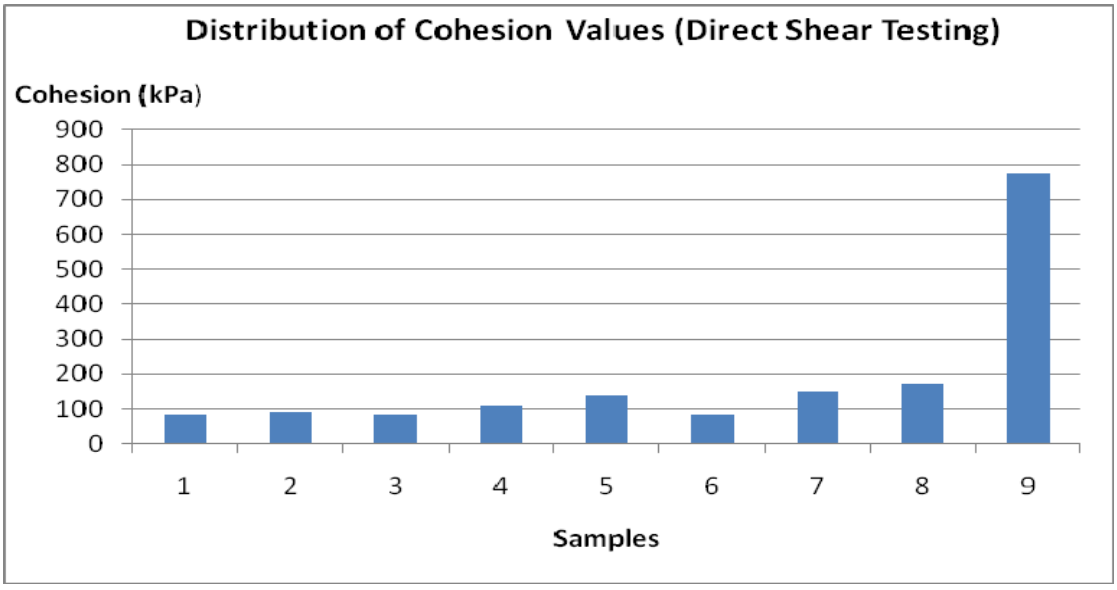


Figure 4.3.2 –Direct Shear Cohesion Data for Lithology A

Statistical Summary Friction (Deg)	
Mean	34.56
Median	35.00
Standard Deviation	9.01
Coefficient of Variability	26.07
Number of Samples	9.00
RMVI	289.67

Table 4.3.3 – Distribution of Friction Angle Values from Direct Shear Testing

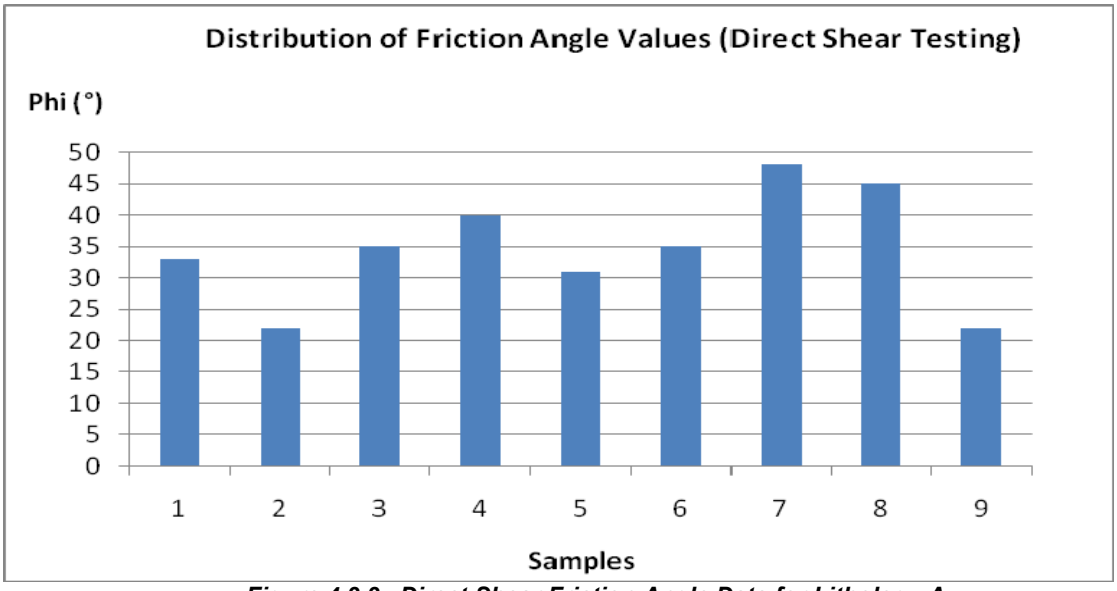


Figure 4.3.3 –Direct Shear Friction Angle Data for Lithology A

Triaxial Data

Statistical Summary Cohesion (kPa)	
Mean	350.00
Median	320.00
Standard Deviation	236.43
Coefficient of Variability	67.55
Number of Samples	3.00
RMVI	**2251.73**

Table 4.3.4 – Distribution of Cohesion Values from Triaxial Testing

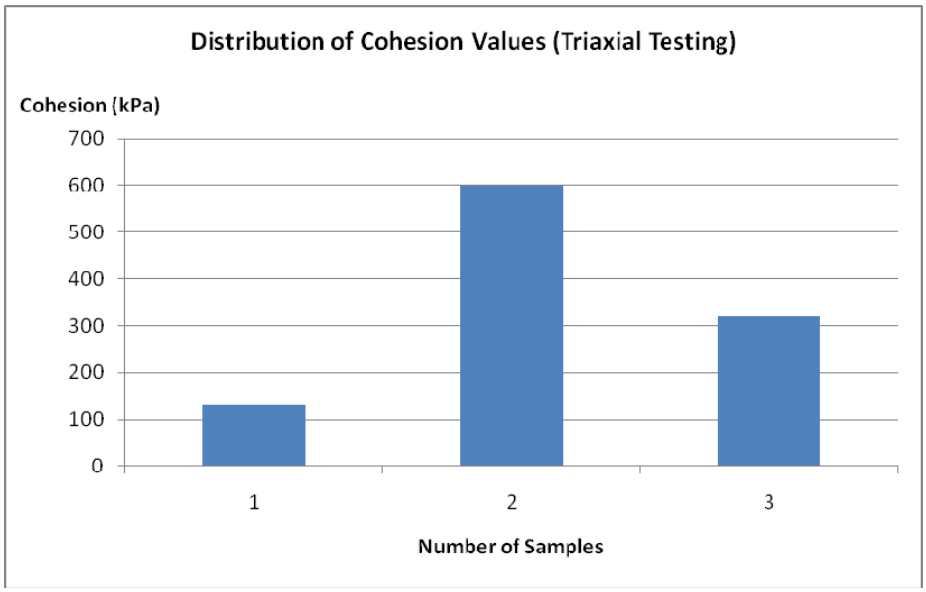


Figure 4.3.4 –Triaxial Cohesion Data for Lithology A

Statistical Summary Friction Angle	
Mean	39.16
Median	37.00
Standard Deviation	6.04
Coefficient of Variability	15.44
Number of Samples	3
RMVI	514.76

Table 4.3.5 – Distribution of Friction Angle Values from Triaxial Testing

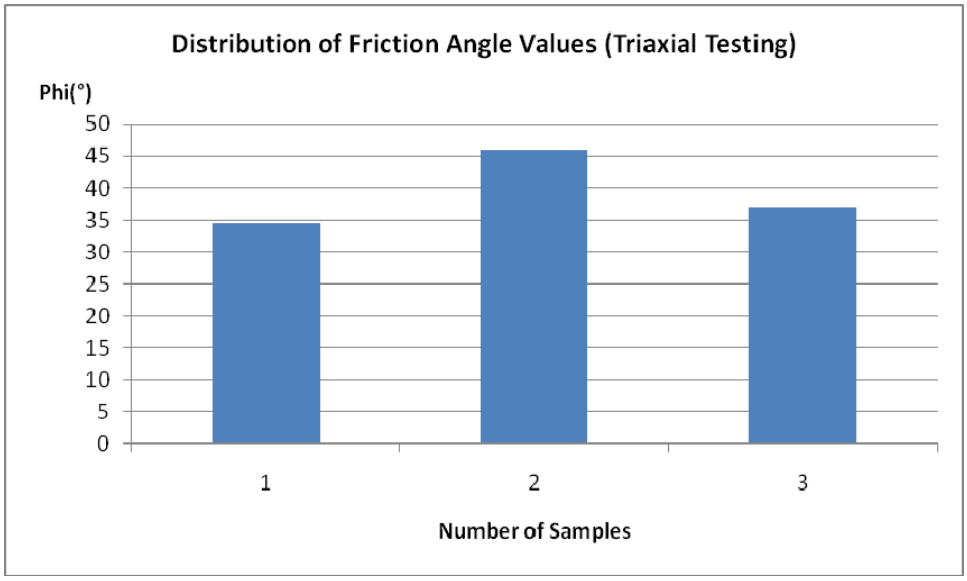


Figure 4.3.5 –Triaxial Friction Angle Data for Lithology A

5 CASE STUDY B

The secondary geological unit that was assessed as part of Case Study B is a very weak highly weathered sedimentary shale unit; which does not contain a pervasive foliation. In general this unit is significantly weaker than the rock unit assessed as part of case study A; and is generally more variable, i.e. there is a higher degree of variability in its strength and mechanical parameters.

5.1 Statistical Summary of Empirical Data

Geotechnical Parameter	Mean	Mode/ Modal Class	Median	Standard Deviation	Coefficient of Variability	RMVI
Strength Estimates (MPa)	4.40	0.25	1.75	6.60	150.00	9.63
JRC	8.90	4.00	10.00	6.80	76.40	3.69
Infill Width (mm)	0.50	0.00	0.10	0.60	120.00	5.80
RQD (%)	23.40	0.00-10.00	0.00	30.60	130.77	12.97
RMR_{89}	31.60	25.00-30.00	29.00	8.50	26.90	2.67

Table 5.1.1 Statistical Summary of Empirical Data

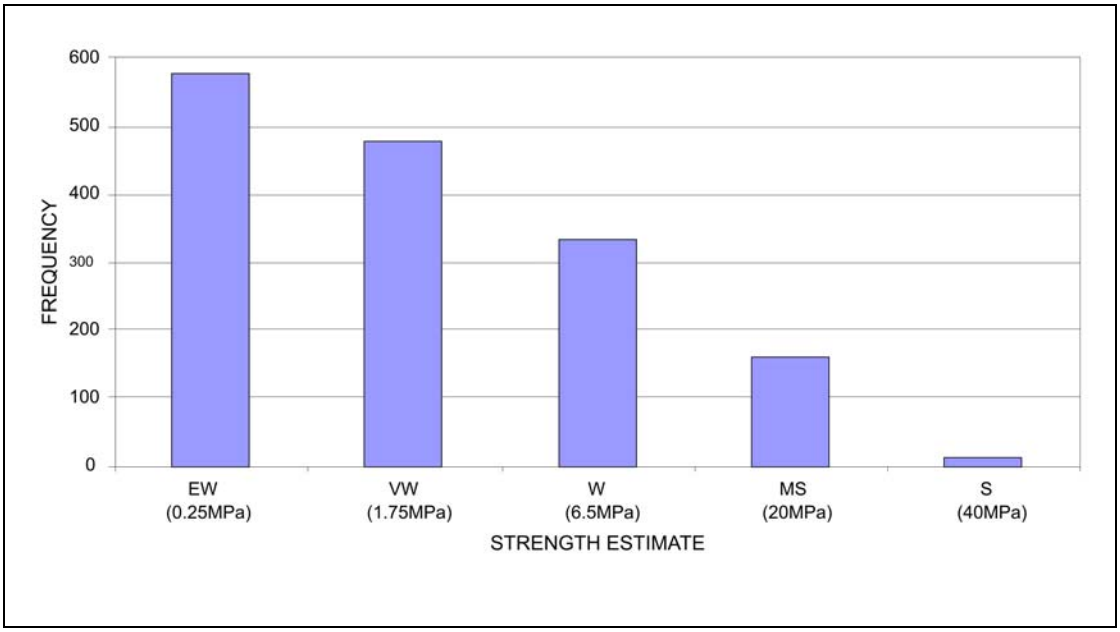


Figure 5.1.1 Strength Estimate for Lithology B

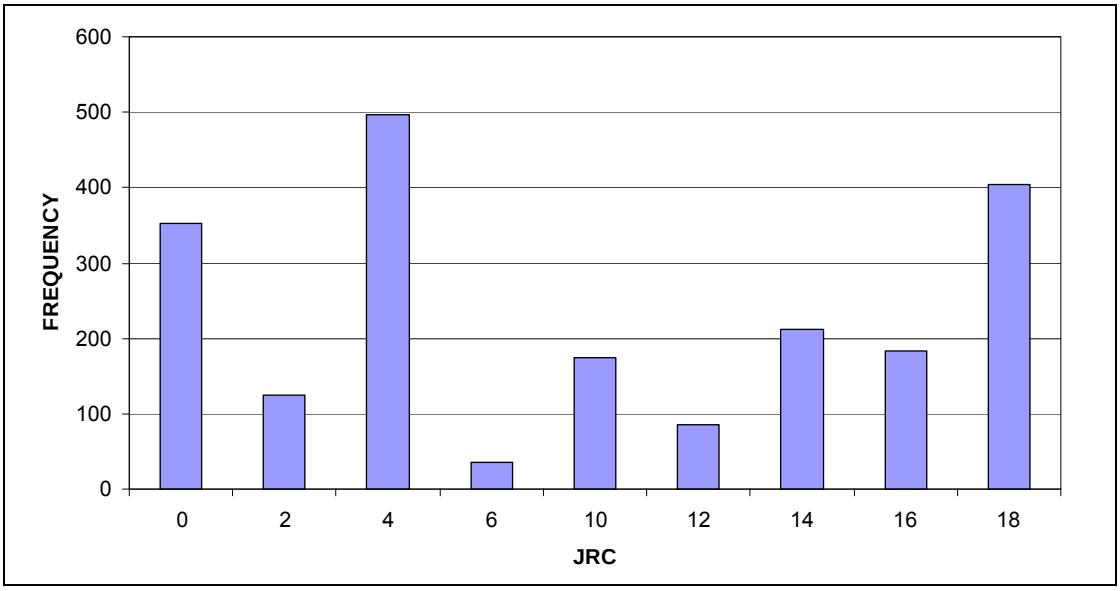


Figure 5.1.2 JRC for Lithology B

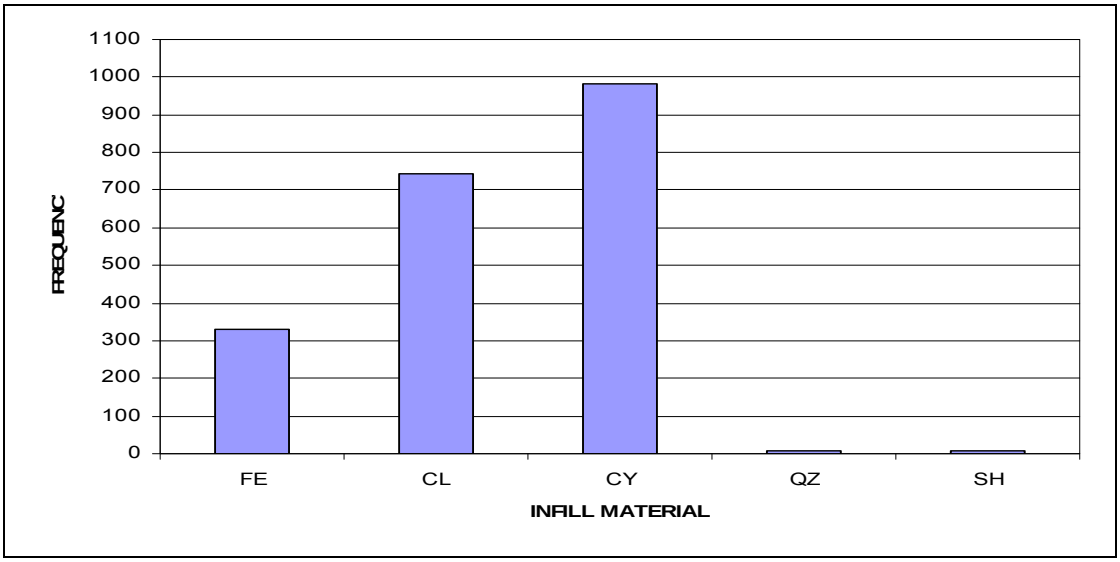


Figure 5.1.3 Infill Type for Lithology B

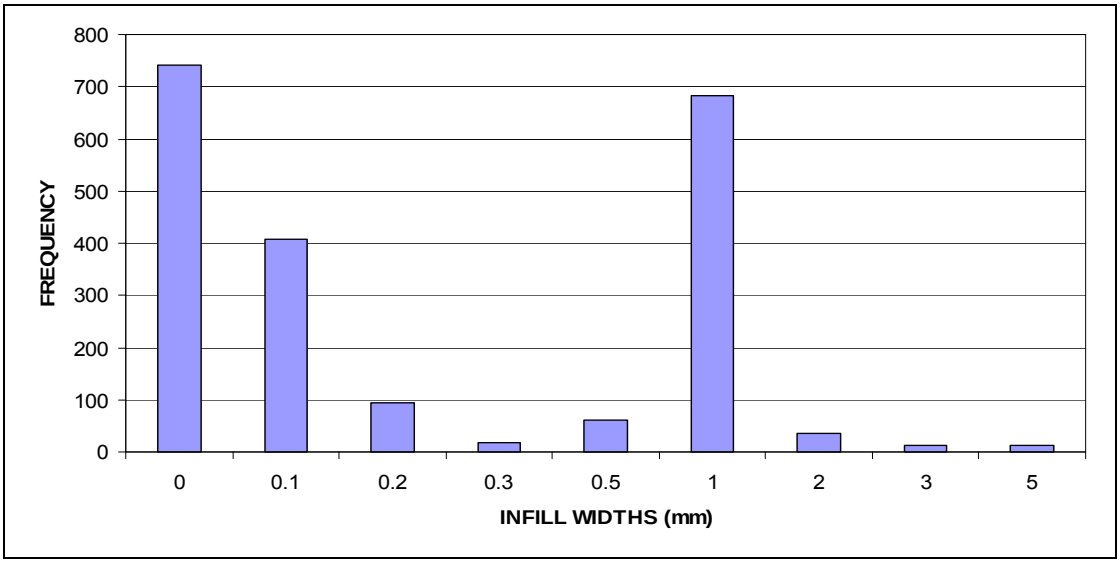


Figure 5.1.4 Infill Widths for Lithology B

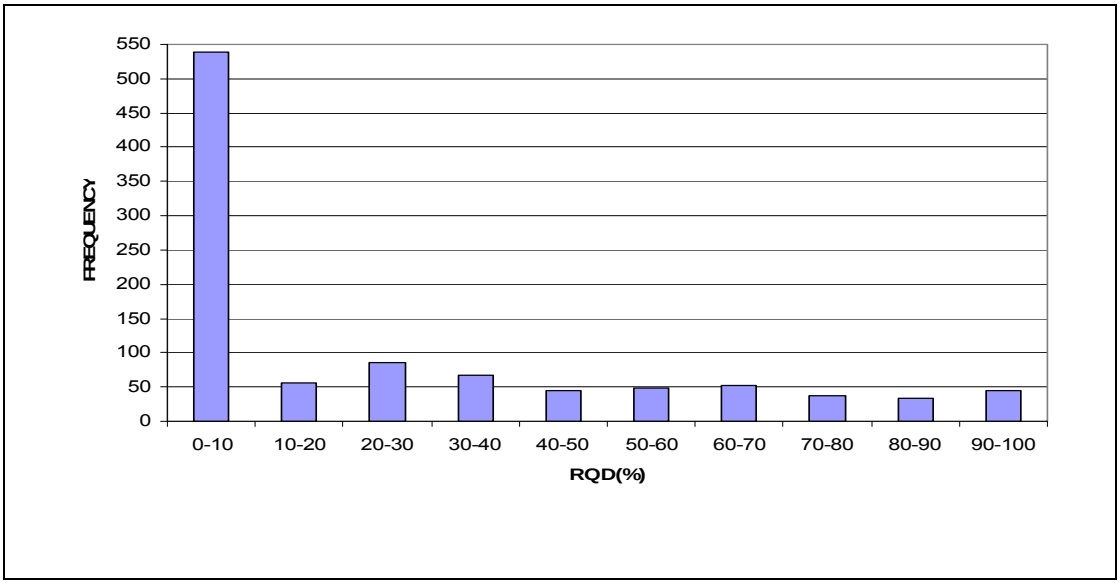


Figure 5.1.5 RQD for Lithology B

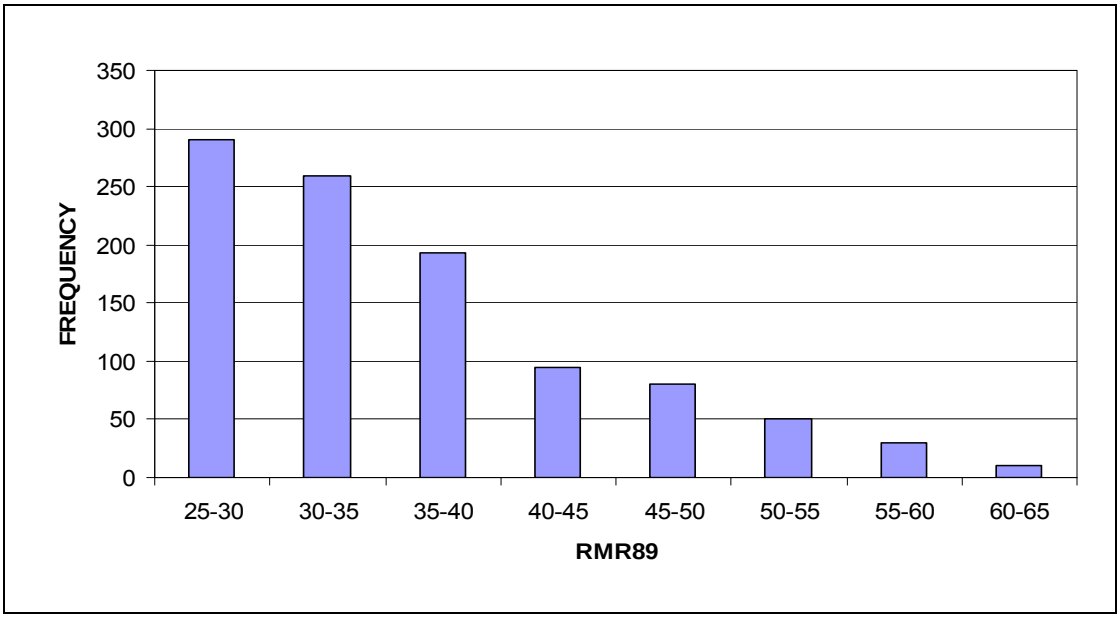


Figure 5.1.6 RMR_{89} for Lithology B

5.2 Statistical Summary of Analytical Data

UCS

Statistical Summary	
Mean	4.40
Median	2.24
Standard Deviation	4.77
Coefficient of Variability	108.48
Number of Samples	12.00
RMVI	903.98

Table 5.2.1 Statistical Summary of UCS Data

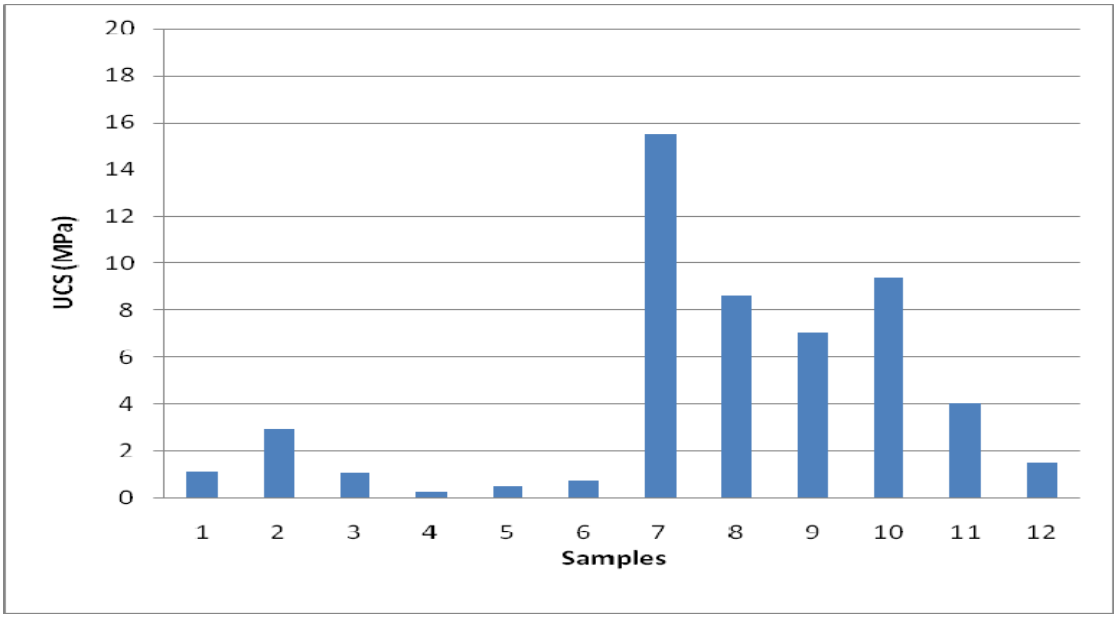


Figure 5.2.1 –UCS Data for Lithology B

Direct Shear Data

Statistical Summary Cohesion (kPa)	
Mean	53.75
Median	45
Standard Deviation	35.43
Coefficient of Variability	65.92
Number of Samples	8
RMVI	823.98

Table 5.2.2 Statistical Summary of Direct Shear Cohesion Data

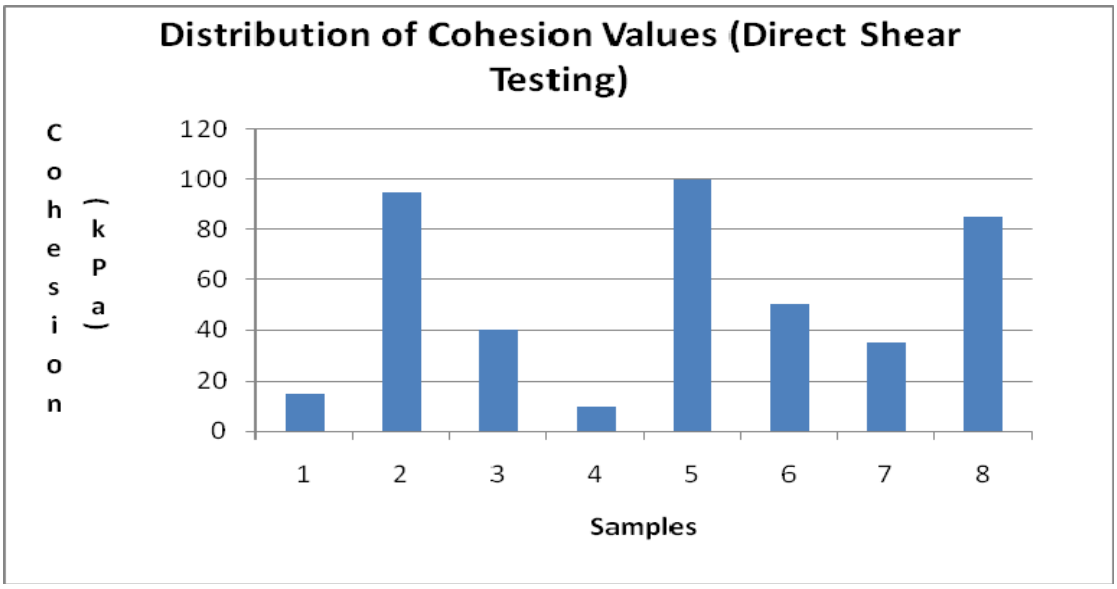


Figure 5.2.2 –Direct Shear Cohesion Data for Lithology B

Statistical Summary Friction Angle (Deg)	
Mean	21.15
Median	21
Standard Deviation	8.51
Coefficient of Variability	40.28
Number of Samples	8
RMVI	503.52

Table 5.2.3 Statistical Summary of Direct Shear Friction Angle Data

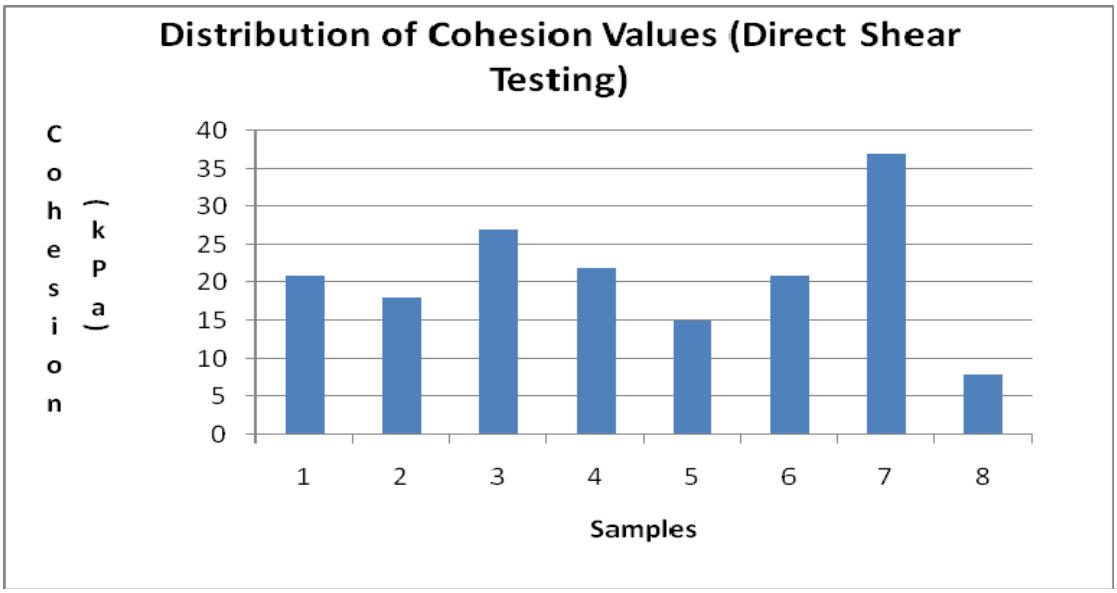


Figure 5.2.3 –Direct Shear Friction Angle Data for Lithology B

Triaxial Data

Statistical Summary Cohesion (kPa)	
Mean	705.29
Median	345.00
Standard Deviation	859.33
Coefficient of Variability	121.84
Number of Samples	7
RMVI	1740.59

Table 5.2.4 Statistical Summary of Triaxial Cohesion Data

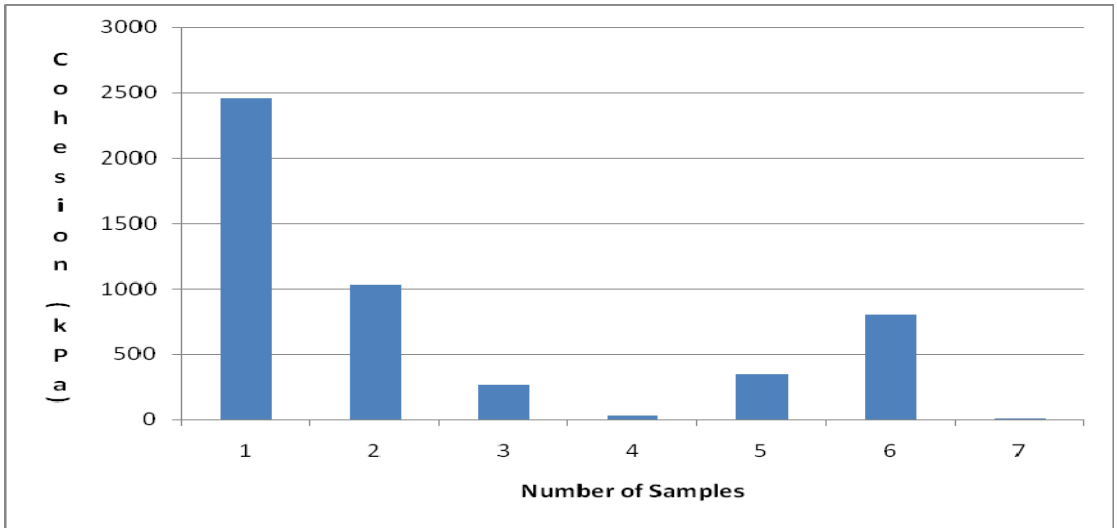


Figure 5.2.4 –Triaxial Test Cohesion Data for Lithology B

Statistical Summary Friction Angle	
Mean	31.60
Median	34.00
Standard Deviation	10.24
Coefficient of Variability	32.42
Number of Samples	7
RMVI	463.13

Table 5.2.4 Statistical Summary of Triaxial Friction Angle Data

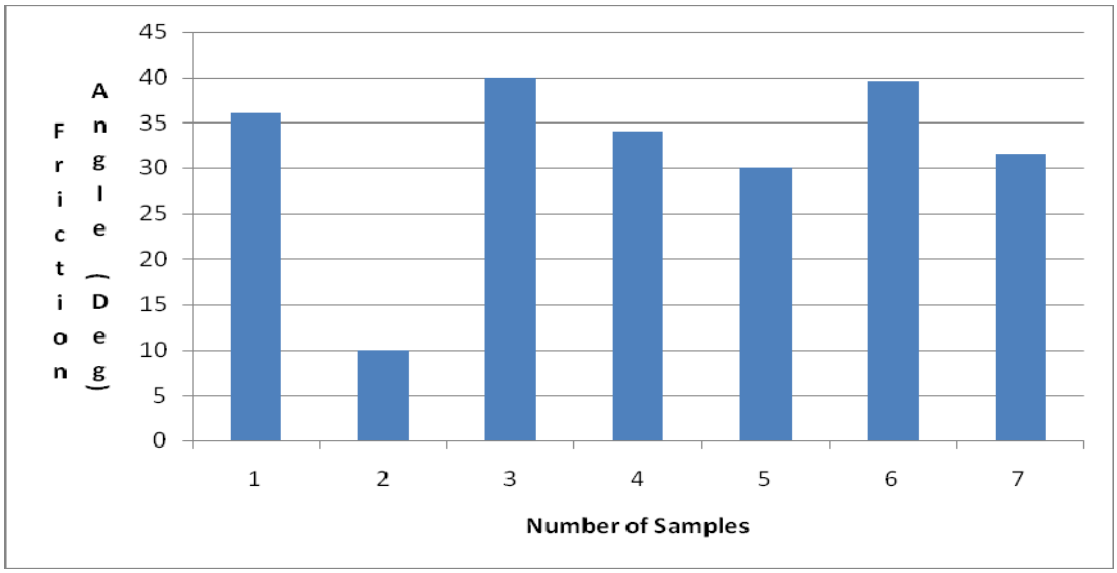


Figure 5.2.5 –Triaxial Test Friction Angle Data for Lithology B

6 CASE STUDY C

The geological unit that was assessed as part of Case Study C is a moderately strong foliated metamorphic rock mass; which contain a pervasive foliation. In general this unit is stronger than the rock units assessed as part of case study A and B.

6.1 Statistical Summary of Empirical Data

Geotechnical Parameter	Mean	Mode/ Modal Class	Median	Standard Deviation	Coefficient of Variability	RMVI
Strength Estimates (MPa) 104 Samples	35.20	1.75	20.00	40.33	114.57	110.17
JRC 381 Samples	12.06	14.00	14.00	4.85	40.22	10.56
Infill Width (mm) 381 Samples	4.71	0.50	0.50	23.35	495.75	130.12
RQD (%) 104 Samples	59.91	90.00-100.00	66.55	29.36	49.01	47.12
RMR_{89} 104 Samples	46.53	50.00-55.00	47.00	12.77	27.44	26.39

Table 6.1.1 Statistical Summary of Core Logging Data (Lithology C)

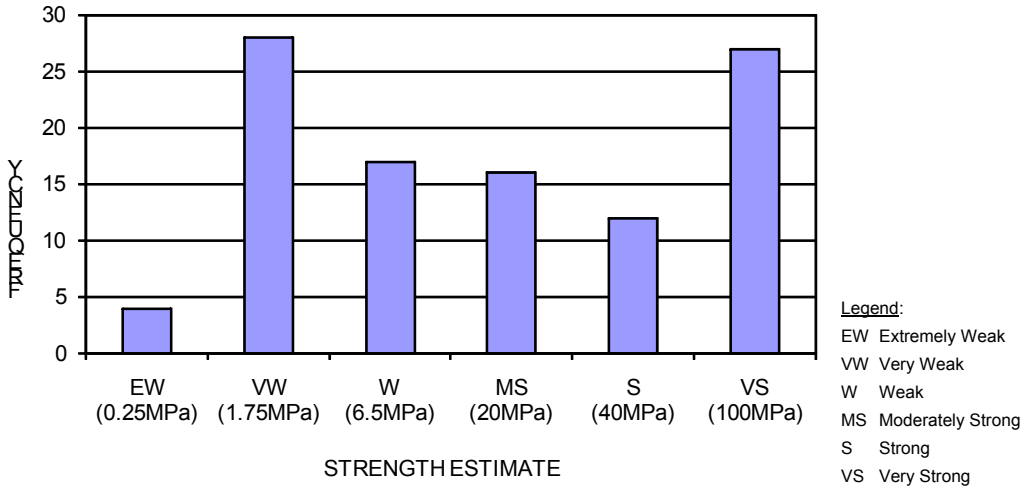


Figure 6.11 Strength Estimate for Lithology C

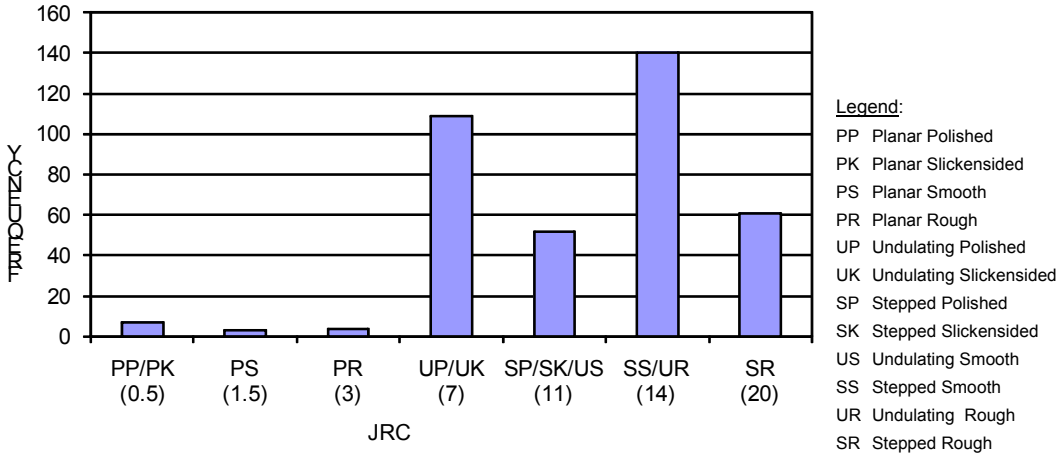


Figure 6.12 JRC for Lithology C

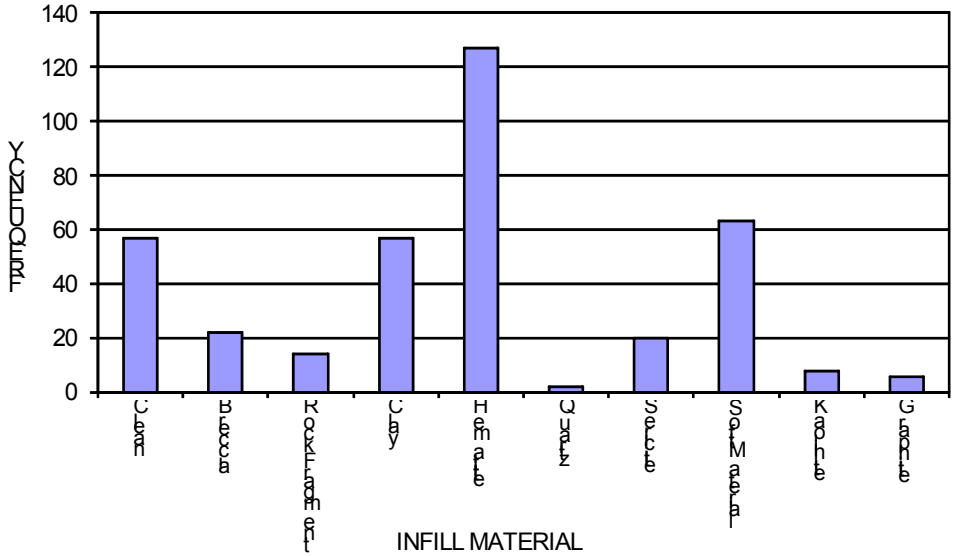


Figure 6.13 Infill Type for Lithology C

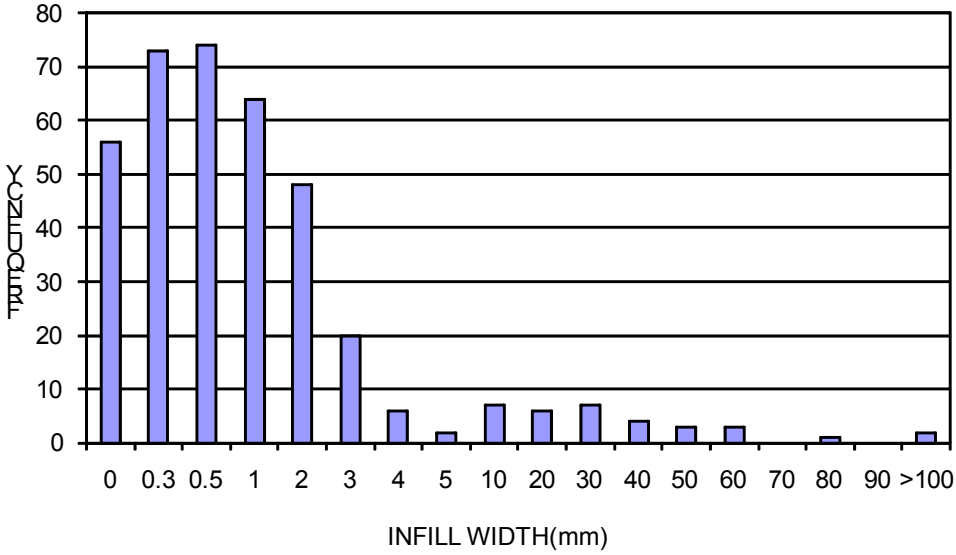


Figure 6.1.4 Infill Width for Lithology C

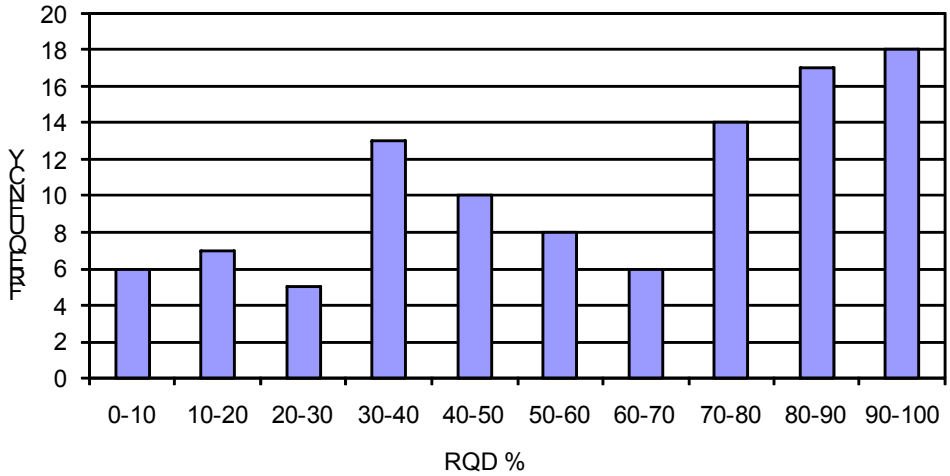


Figure 6.15 RQD for Lithology C

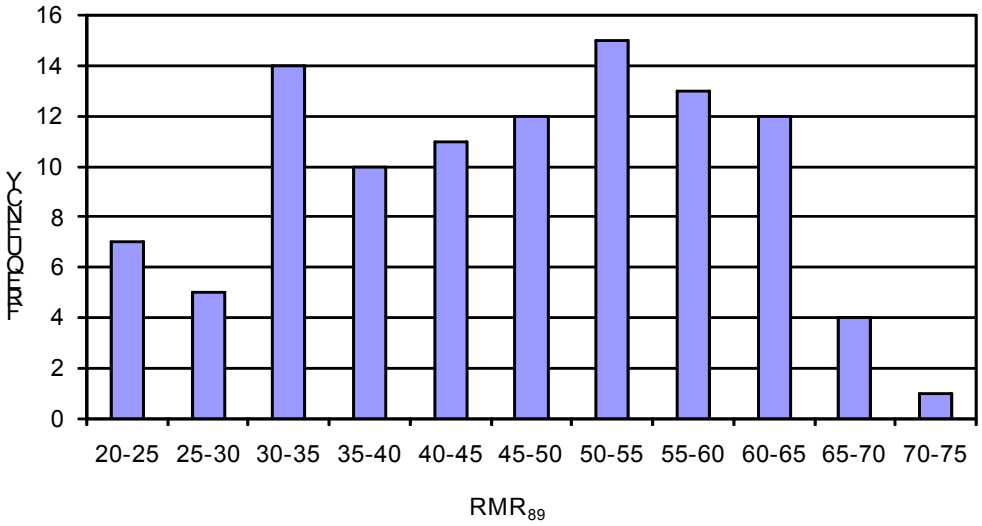


Figure 6.16 RMR_{89} for Lithology C

6.2 Statistical Summary of Analytical Data

UCS

Only 2 UCS tests were done on this lithology, yielding 106 MPa and 143MPa respectively.

Direct Shear

Statistics	
Mean	22.5
Median	22.5
Mode/Modal Class	N/A
Standard Deviation	6.45
Samples	4
Coefficient of Variability	28.67
RMVI	716.67

Table 6.2.1 – Direct Shear Cohesion Statistics for Lithology C

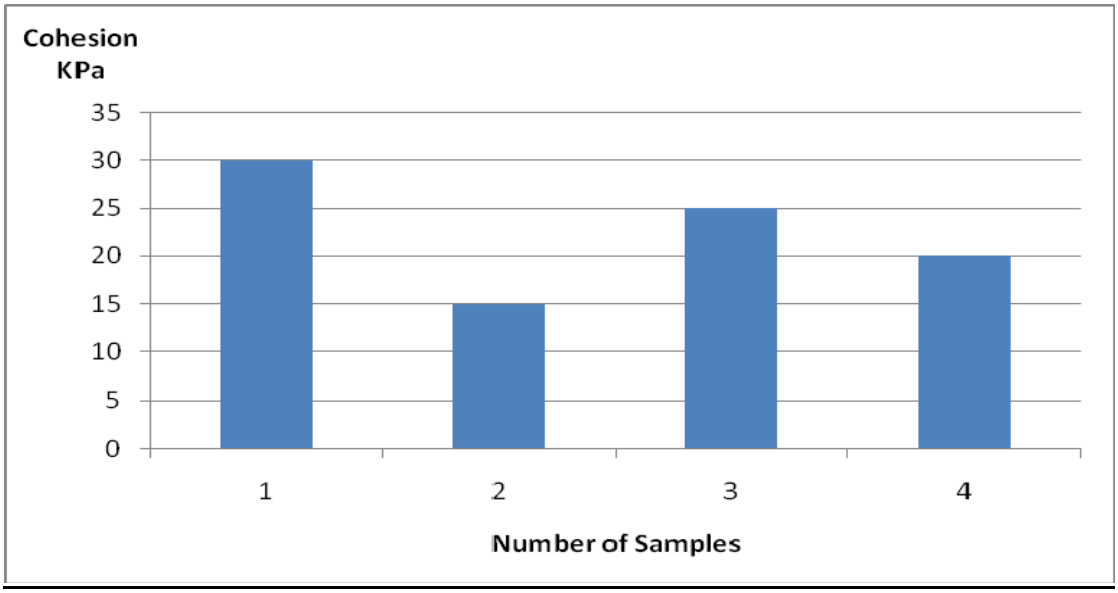


Figure 6.2.1 – Direct Shear Cohesion for Lithology C

Statistics	UCS, 2009 (MPa)
Mean	29.75
Median	30.5
Mode/Modal Class	N/A
Standard Deviation	5.38
Samples	4
Coefficient of Variability	18.1
RMVI	452.1

Table 6.2.1 – Direct Shear Friction Angle Statistics for Lithology C

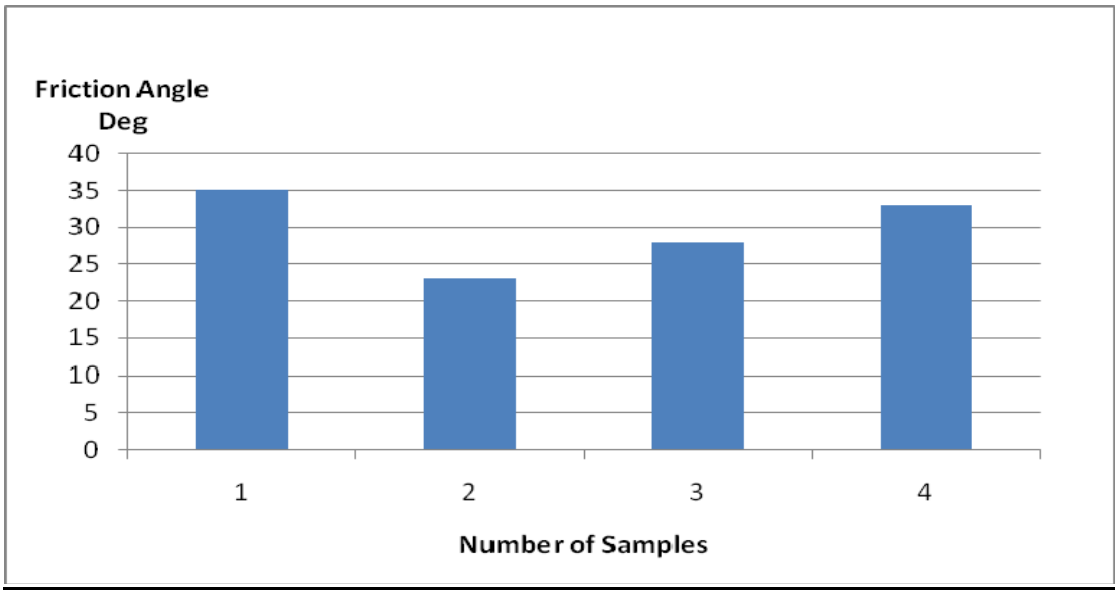


Figure 6.2.1 – Direct Shear Friction Angle for Lithology C

7 CASE STUDY D

The geological unit that was assessed as part of Case Study D is a moderately strong foliated metamorphic rock mass; which contains a pervasive foliation. In general this unit is stronger than the rock units assessed as part of case study A and B, and very similar to that of case study C. The primary difference is that here the foliation planes are infilled with clay material.

7.1 Statistical Summary of Empirical Data

Geotechnical Parameter	Mean	Mode/ Modal Class	Median	Standard Deviation	Coefficient of Variability	RMVI
Strength Estimates (MPa) 189 Samples	30.07	6.50	20.00	28.83	95.88	50.72
JRC 583 Samples	12.22	14.00	14.00	4.50	36.82	6.31
Infill Width (mm) 583 Samples	8.23	0.30	0.50	33.90	411.91	70.65
RQD (%) 189 Samples	62.82	90.00-100.00	69.70	29.86	47.53	25.14
RMR_{89} 189 Samples	47.68	50.00-55.00	48.00	9.66	20.26	10.71

Table 7.1.1 Statistical Summary of Core Logging Data (lithology D)

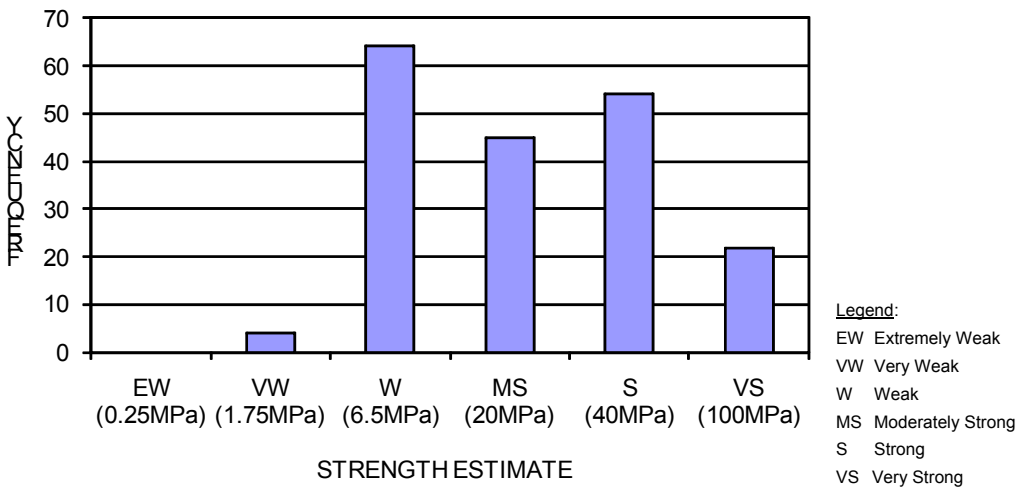


Figure 7.1.1 Strength Estimate for Lithology D

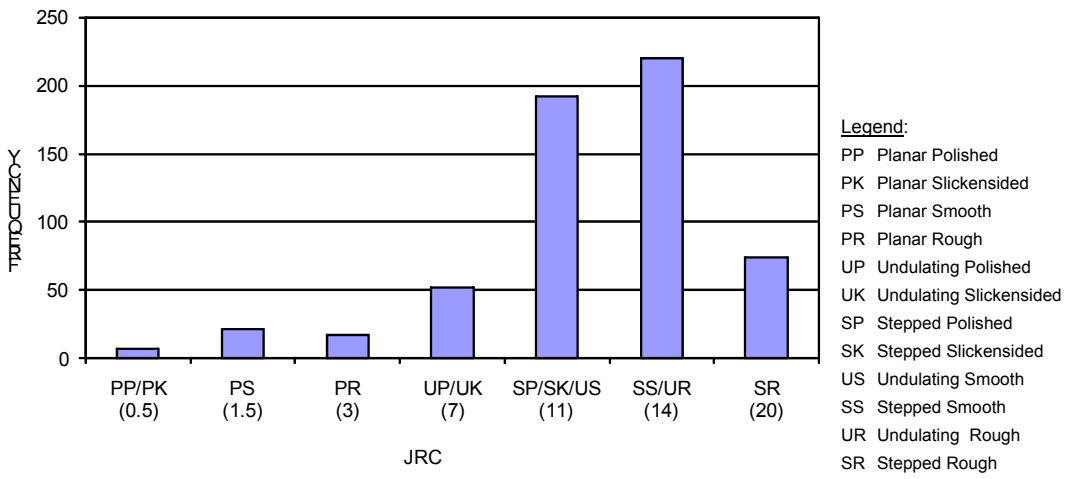


Figure 7.1.2 JRC for Lithology D

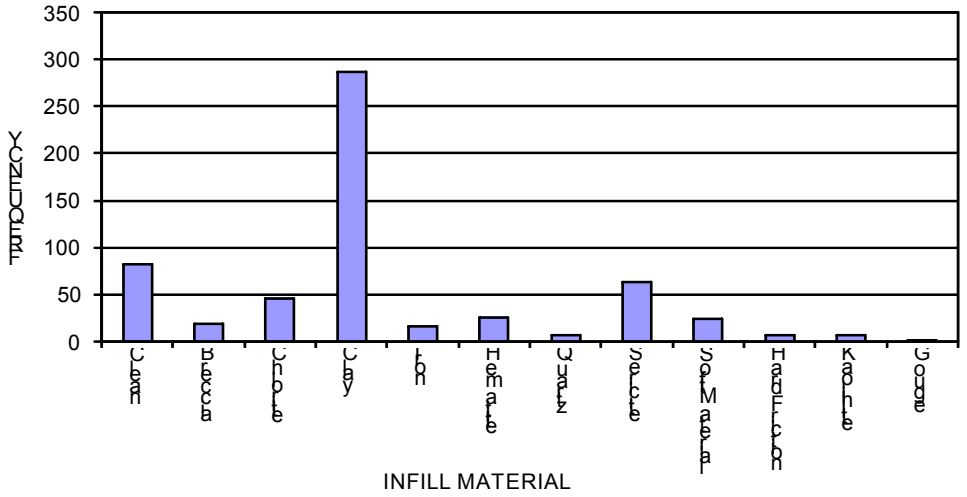


Figure 7.1.3 Infill Material for Lithology D

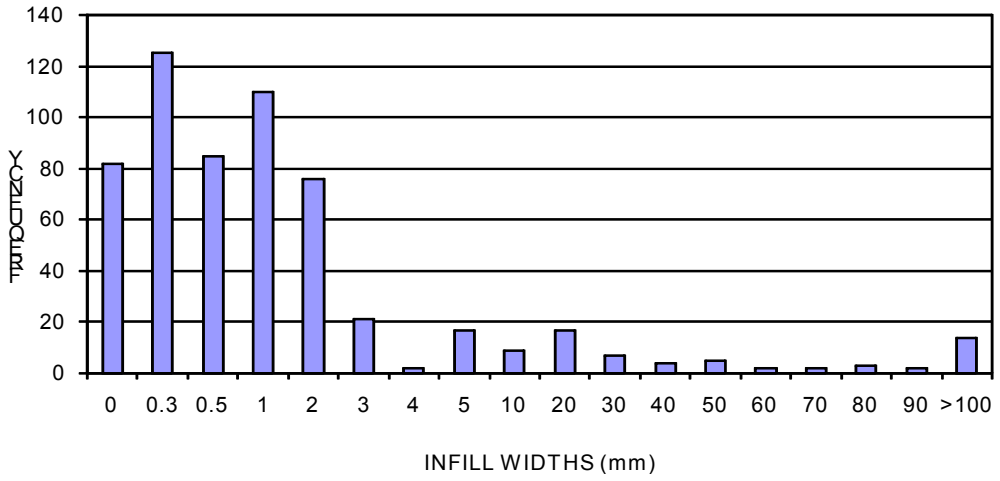


Figure 7.1.4 Infill widths for Lithology D

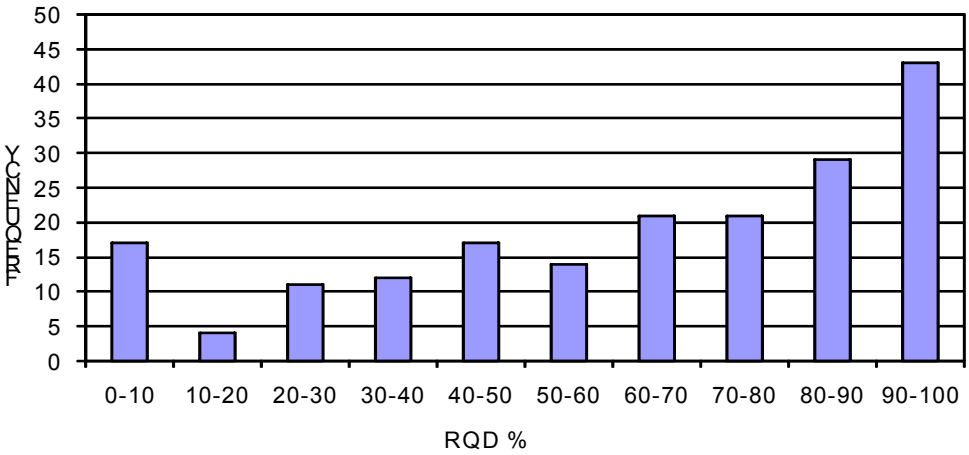


Figure 7.1.5 RQD for Lithology D

Figure 8.3.2_6
RMR89 – Southeast Domain

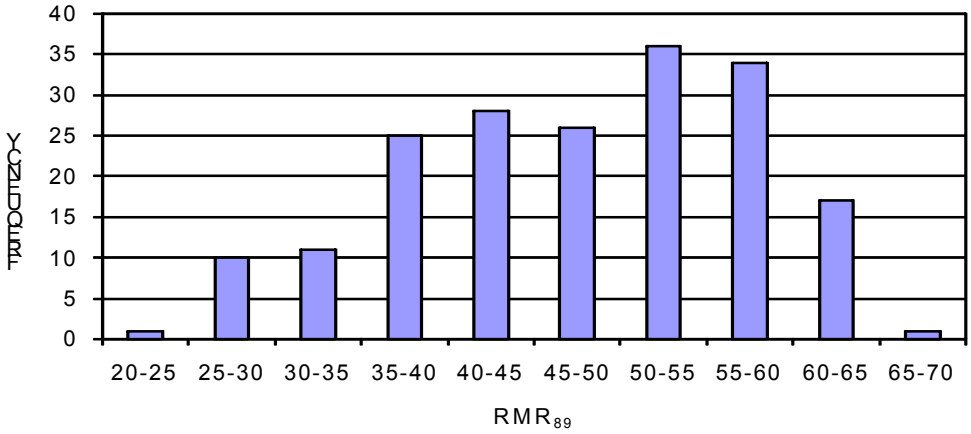


Figure 7.1.6 RMR_{89} for Lithology D

7.2 Statistical Summary of Analytical Data

UCS

Statistics UCS (MPa)	
Mean	30.44
Median	12.20
Mode/Modal Class	0-10
Standard Deviation	40.42
Samples	33
Coefficient of Variability	132.78
RMVI	402.4

Table 7.2.1 – UCS Statistics for Lithology D

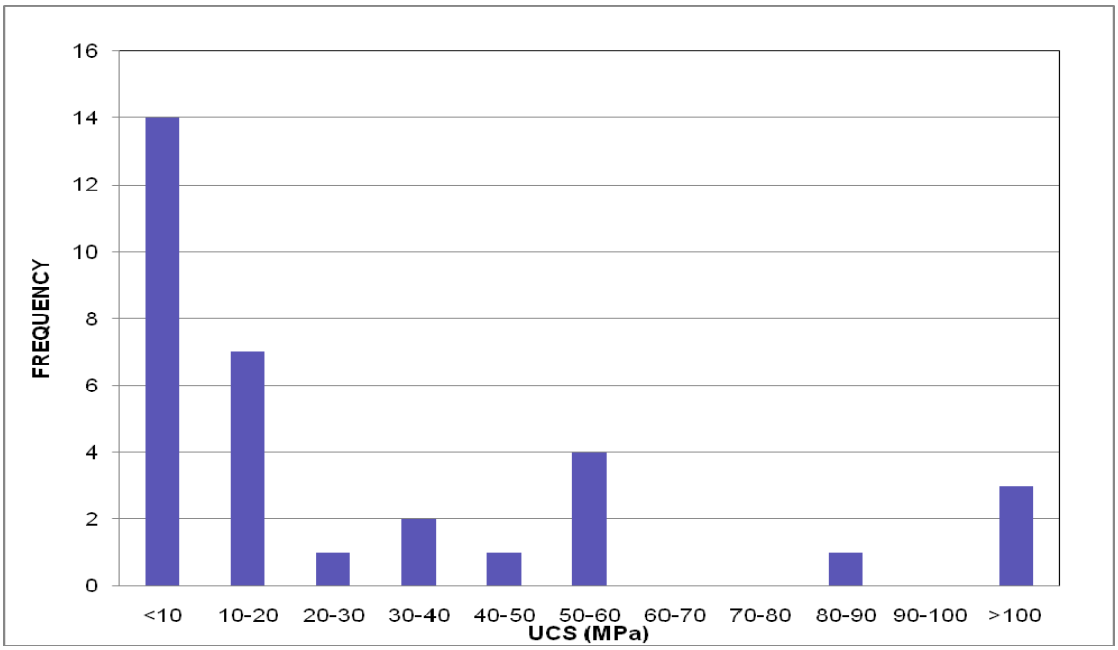


Figure 7.2.1 – UCS for Lithology D

Direct Shear

Statistics Direct Shear Cohesion (KPa)	
Mean	26.5
Median	20.5
Mode/Modal Class	15.0
Standard Deviation	22.61
Samples	10
Coefficient of Variability	85.32
RMVI	853.21

Table 7.2.2 – Direct Shear Cohesion Statistics for Lithology D

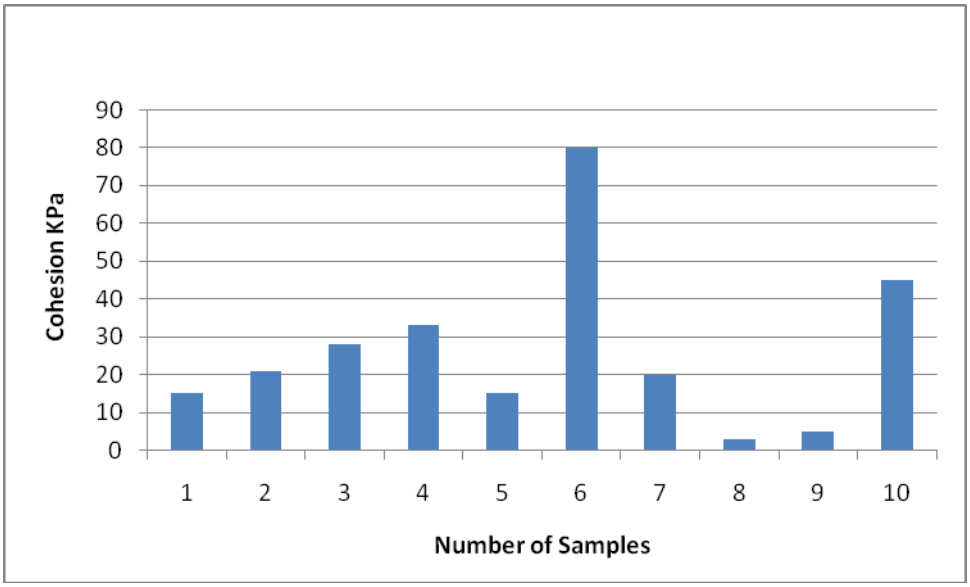


Figure 7.2.2 – Direct Shear Cohesion for Lithology D

Statistics Direct Shear Statistics (KPa)	
Mean	25.2
Median	24.5
Mode/Modal Class	30.0
Standard Deviation	7.46
Samples	10
Coefficient of Variability	29.6
RMVI	296.03

Table 7.2.3 – Direct Shear Friction Angle Statistics for Lithology D

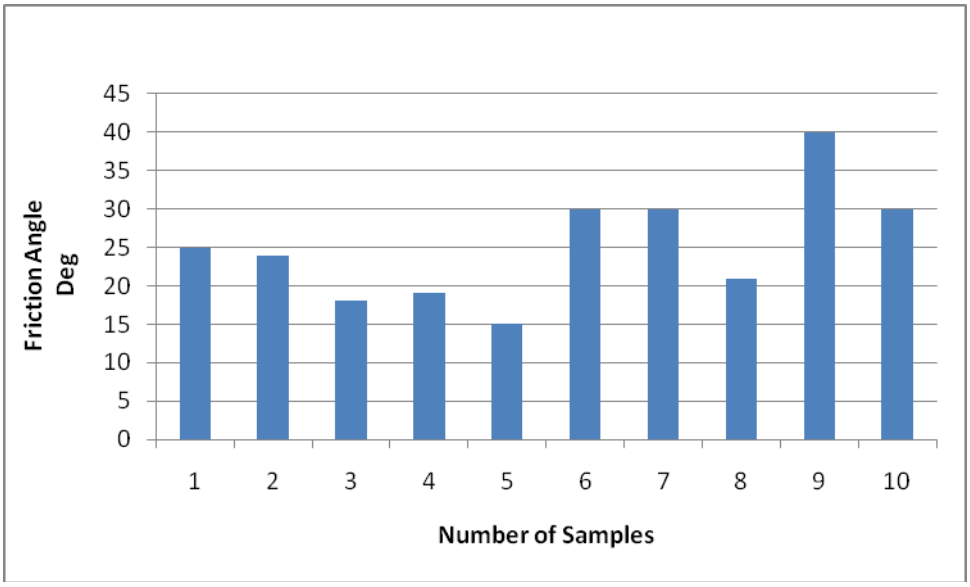


Figure 7.2.3 – Direct Shear Friction Angles for Lithology D

8 ASSESSMENT OF ROCK MASS SHEAR STRENGTH PARAMETERS

8.1 Lithology A

The author utilised the RMR_{89} values as calculated from core logging, within the software package Roclab to empirically assess material properties using the following input parameters and relationship:

- **Disturbance Factor** : 0.5 (As moderate amount of exfoliation is anticipated);
- **Geological Strength Index** (GSI) = RMR_{89} – 5 : 36 (Mean RMR_{89} values minus 5);
- **Unconfined Compressive Strength** (UCS) : 50 MPa;

Based on the above input parameters, the following material parameters have been assessed, at a confinement of 1.5MPa as shown in Fig 6.1.1:

- Cohesion – 550 kPa; and
- Friction Angle – 40.7°

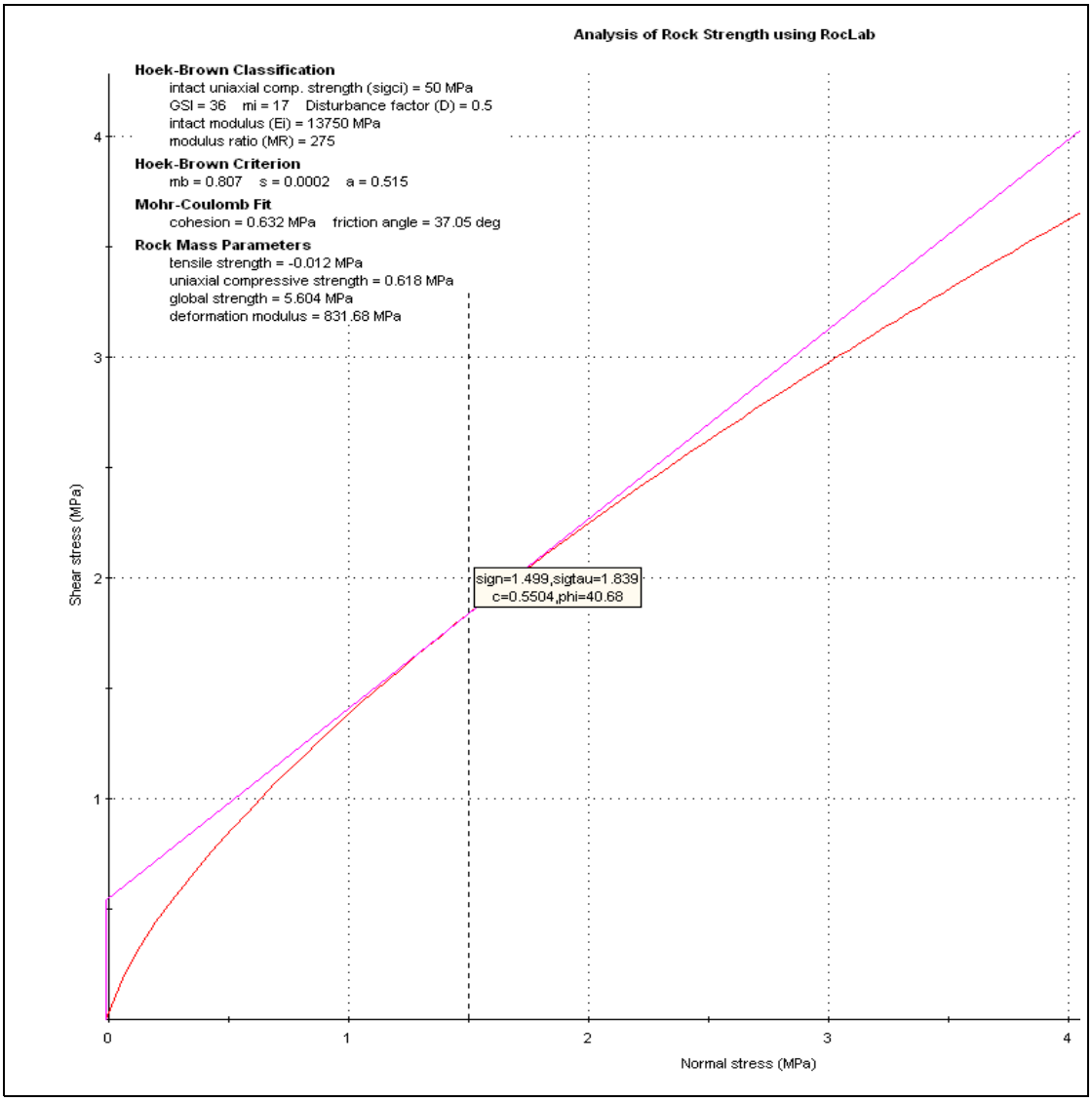


Figure 8.1.1 Empirical Assessment of Lithology A (Rock Mass) Shear Strength

8.2 Lithology B

The author utilised the RMR_{89} values calculated from core logging, within the software package Roclab, to empirically assess material properties using the following input parameters:

- **Disturbance Factor** : 0.3 (As only small amounts of exfoliation is expected);
- **Geological Strength Index** (GSI) = RMR_{89} – 5 : 27 (Mean RMR_{89} values minus 5);
- **Unconfined Compressive Strength** (UCS) : 7.8 MPa;

Based on the above input parameters the following material parameters have been assessed at a confinement of 1.5MPa as shown in Fig 3.28:

- Cohesion – 268 kPa; and
- Friction Angle – 17.2°

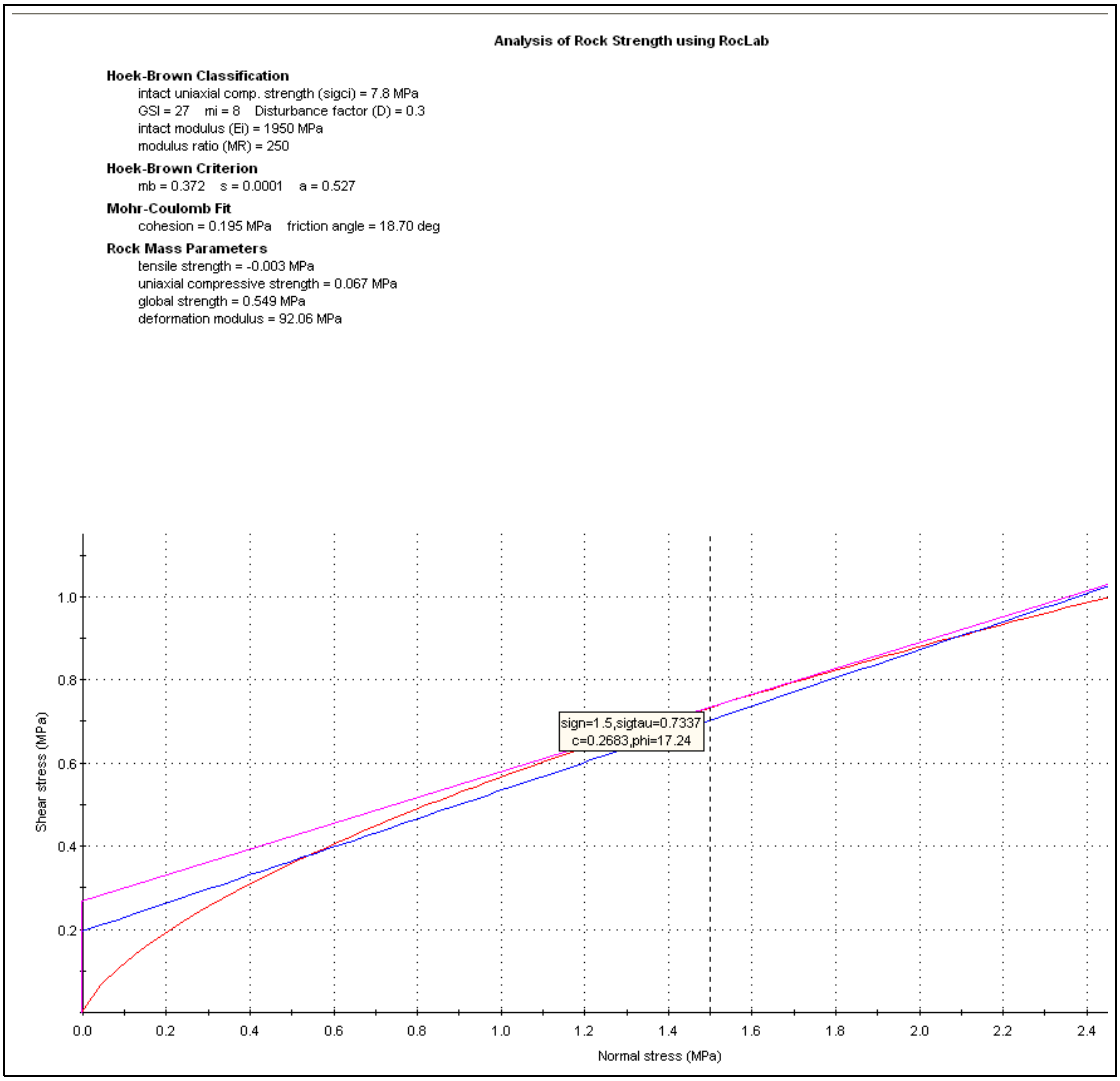


Figure 8.2.2 Empirical Assessment of Lithology B (Rock Mass) Shear Strength

8.3 Lithology C

The author utilised the RMR_{89} values calculated from core logging, within the software package Roclab, to empirically assess material properties using the following input parameters:

- Disturbance Factor = 0.5 (Assuming Good Blasting; i.e. pre-splitting);
- Geological Strength Index (GSI) = RMR_{89} – 5 = 30 (Mean RMR_{89} values minus 5);
- Unconfined Compressive Strength (UCS) = 25MPa;
- Unit Weight = 30KN/m³; and
- Slope Height = 270m.

Based on the above input parameters, the following material parameters have been assessed, at a confinement of 1.5MPa:

- Cohesion – 740kPa; and
- Friction Angle – 24.76°

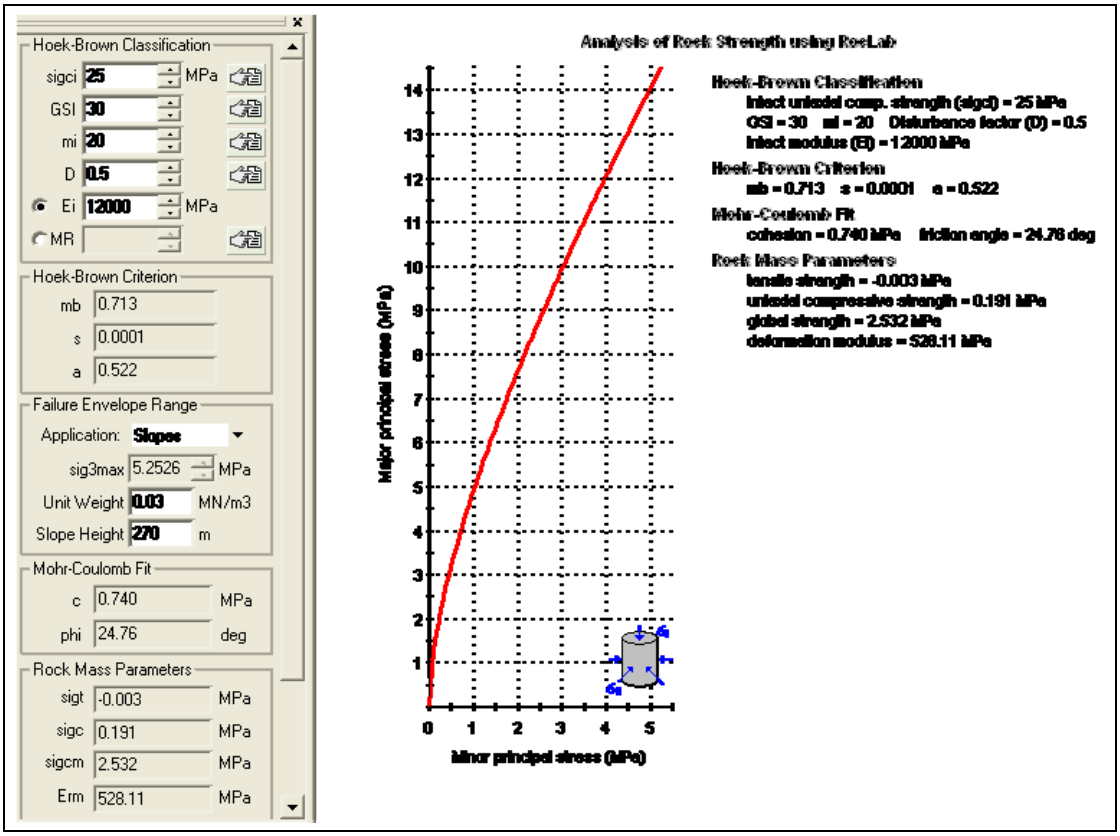


Figure 8.3.1 - Empirical Assessment of Lithology C (Rock Mass) Shear Strength

8.4 Lithology D

The author utilised the RMR_{89} values calculated from core logging, within the software package Roclab, to empirically assess material properties using the following input parameters:

- Disturbance Factor = 0.5 (Assuming Good Blasting; i.e. pre-splitting);
- Geological Strength Index (GSI) = RMR_{89} – 5 = 43 (Mean RMR_{89} values minus 5);
- Unconfined Compressive Strength (UCS) = 32.5MPa;
- Unit Weight = 27KN/m³; and
- Slope Height = 270m

Based on the above input parameters, the following material parameters have been assessed, at a confinement of 1.5MPa:

- Cohesion – 576kPa; and
- Friction Angle – 21.53°

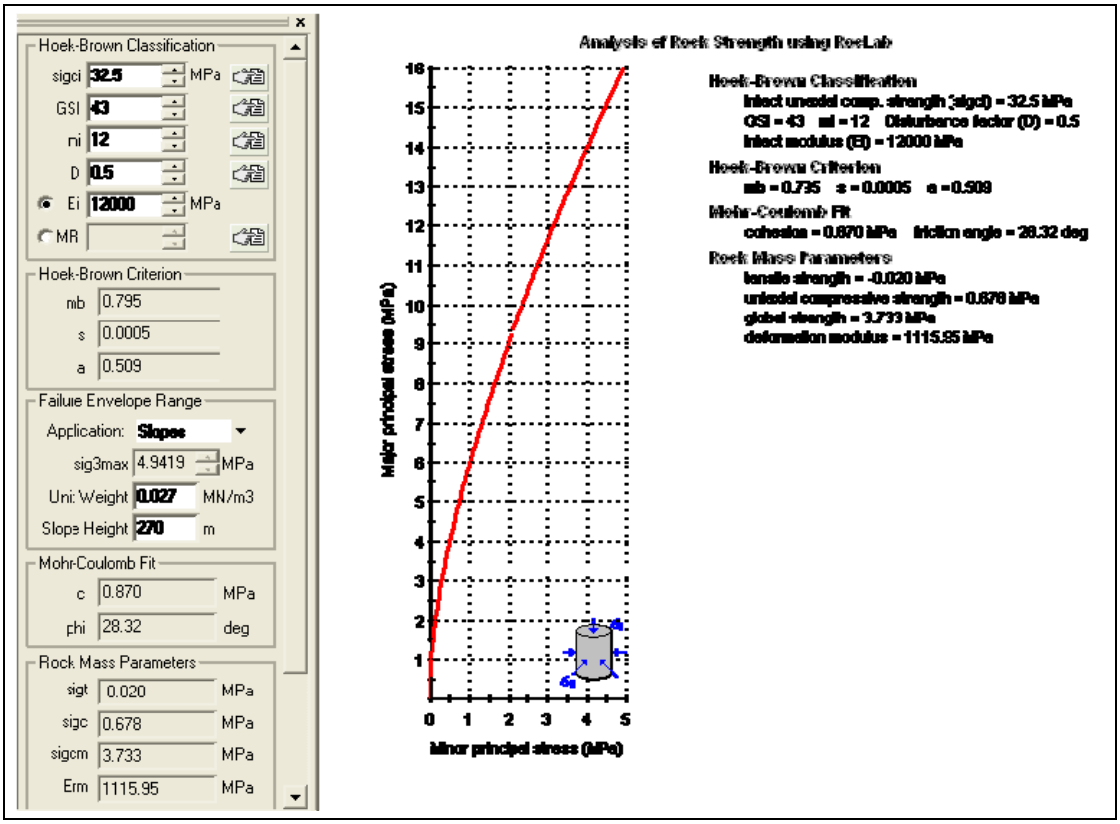


Figure 8.4.1 - Empirical Assessment of Lithology D (Rock Mass) Shear Strength

8.5 Assessment of Bedding Plane Shear Strength

<u>Empirical Assessment of Bedding Plane Shear Strength (Lithology A)</u>

Bedding plane shear strengths were assessed empirically using the Barton – Bandis (1974) criteria and the following values of its components:

- JRC – 10.0 (Assuming silty clay infilling);
- Joint Wall Compressive Strength (JCS) – 50MPa;
- Residual Friction Angle (From triaxial testing) – 30°; and
- Instantaneous shear strength evaluated at batter scale confinement (300KPa).

Based on the above parameters, the following shear strength parameters have been evaluated for the defect planes,:

- Cohesion – 85 kPa, Friction Angle – 45°

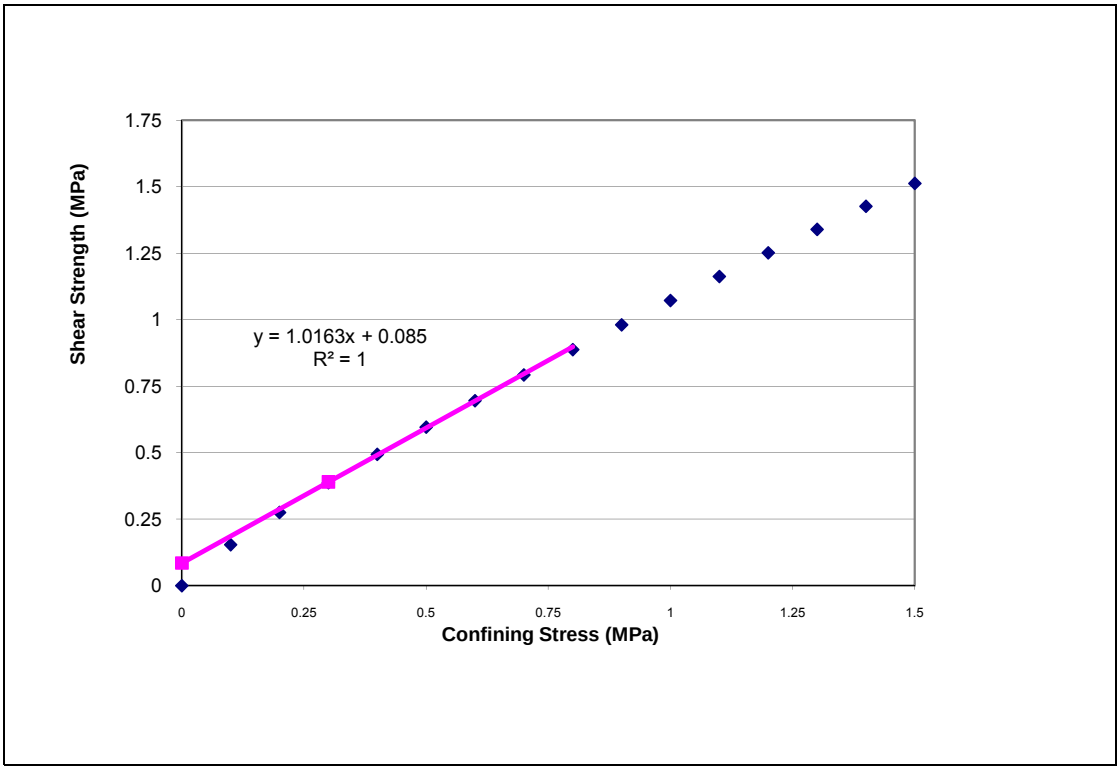


Figure 8.5.1 Barton Bandis Shear Strength Envelope for Lithology A

<u>Laboratory Assessment of Bedding Plane Shear Strength Lithology A</u>

The following parameters have been assessed from direct shear testing on the defect planes:

- Cohesion – 179 kPa; and
- Friction Angle – 35°

The figure below shows the comparison of the empirically assessed material properties and the laboratory assessed values for the defect plane shear strengths.

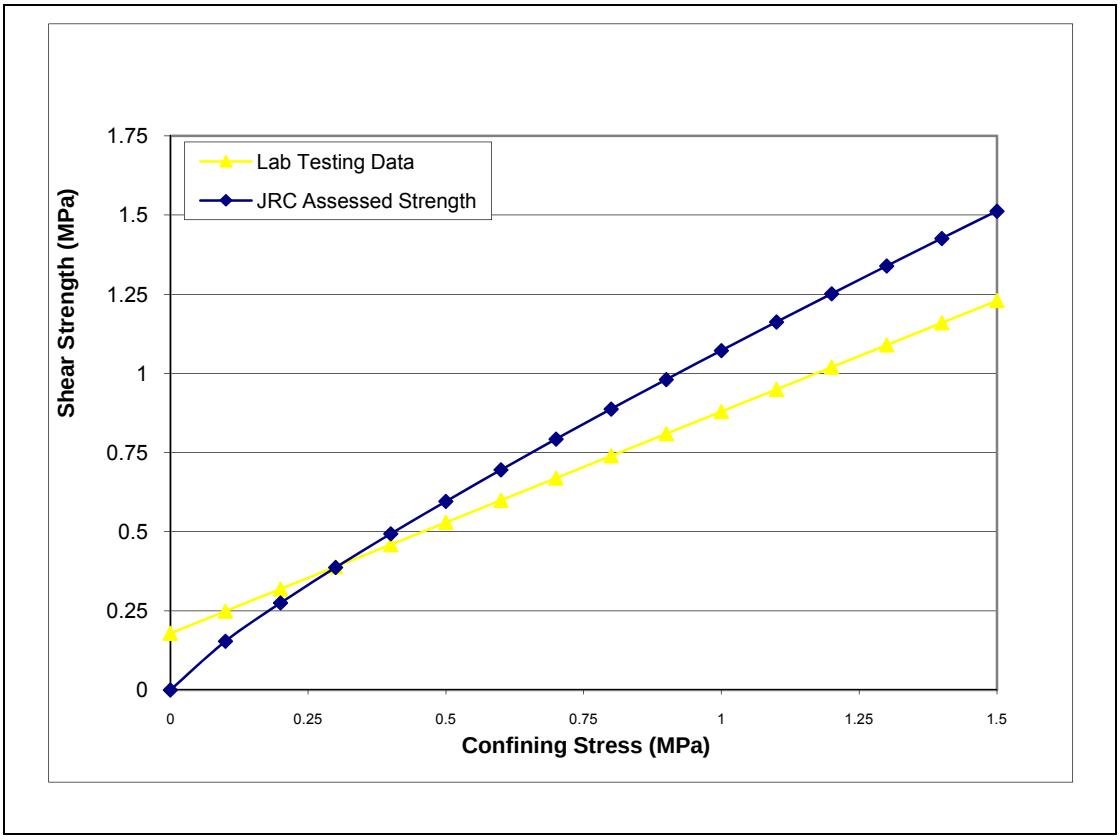


Figure 8.5.2 Comparison of Lithology A Shear Strengths

Empirical Assessment of Bedding Plane Shear Strength (Lithology C)

Defect plane shear strengths were assessed empirically using the Barton – Bandis (1974) criteria for Lithology C. The following inputs were used:

- JRC – 11;
- Joint Wall Compressive Strength (JCS) – 25MPa;
- Residual Friction Angle– 20°; and
- Instantaneous shear strength evaluated at batter scale confinement (~ 1MPa).

Based on the above values, the following shear strength parameters have been evaluated for defect planes, as presented below:

- Cohesion – 120kPa
- Friction Angle – 31°

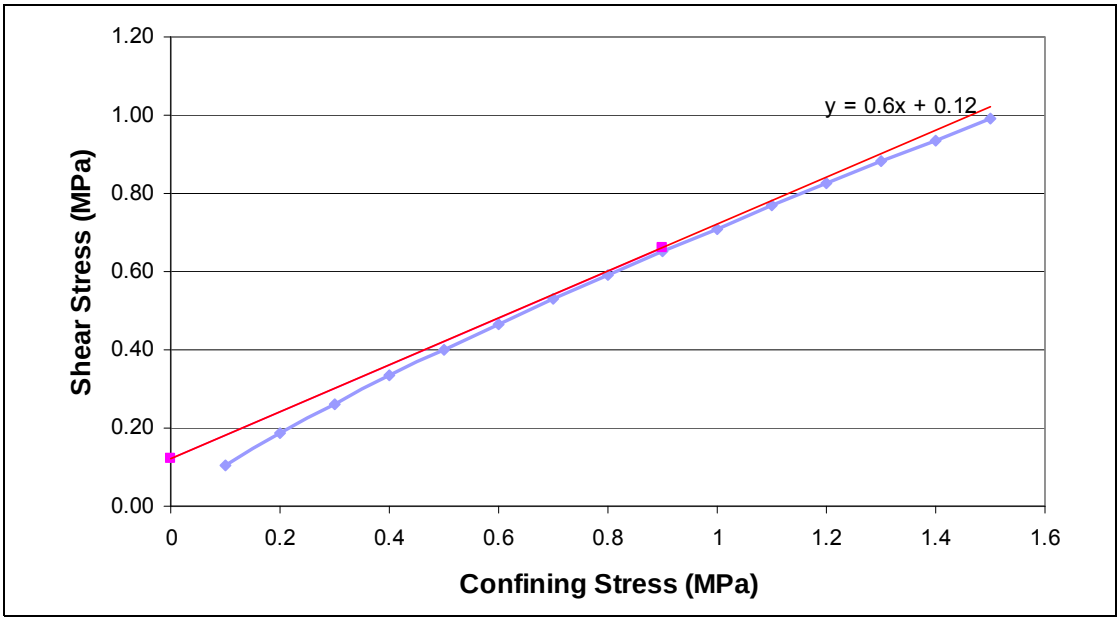


Figure 8.5.3 Barton Bandis Shear Strength Envelope for the Lithology C

Empirical Assessment of Bedding Plane Shear Strength (Lithology D)

Defect plane shear strengths were assessed empirically using the Barton – Bandis (1974) criteria for the Southwest Domain and the following values of its components:

- JRC – 12.24;
- Joint Wall Compressive Strength (JCS) – 33MPa;
- Residual Friction Angle – 15°; and
- Instantaneous shear strength evaluated at batter scale confinement (~ 1MPa).

Based on the above parameters, the following shear strength parameters have been evaluated for defect planes, based on Figure 8.5.3:

- Cohesion – 140kPa
- Friction Angle – 34.6°

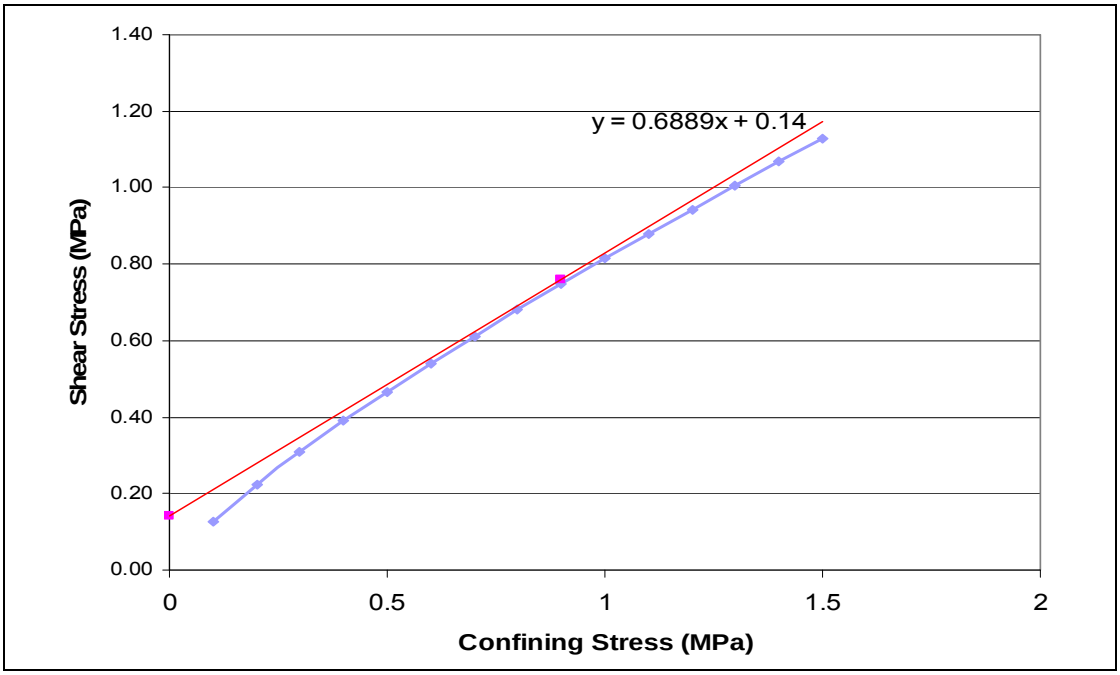


Figure 8.5.3 Barton Bandis Shear Strength Envelope for the Lithology D

9 MATERIAL PROPERTIES USED FOR STABILITY MODELLING

This section details the process by which design strengths can be chosen using the RMVI methodology as being proposed by the author. It is the intent of the author to demonstrate that application of this methodology would facilitate the more appropriate selection of material parameters based on the statistical integrity of the available data.

Typically most slope design engineers choose the most conservative parameters (cohesion and friction angles) determined from empirical or analytical sources. However this process will inherently not be cognisant of the statistical validity and integrity of the data.

Hence choosing the most conservative parameters may not be doing 'justice' to the actual rock mass strength. There it is the author's supposition that the RMVI methodology would provide a more refined methodology for sorting this particular predicament.

This can have a number of implications to the final slope design; which will result in flatter slopes; thereby increasing the volume of waste movement, increased equipment cost to facilitate the larger volumes of waste movement and increased footprint and land clearing requirements.

9.1 Lithology A

Rock Mass Shear Strength

The rock mass shear strength will need to be determined form the available methods as follows:

- **Empirical methods** (i.e. core logging data assessed to determine RMR_{89} values and cohesion and friction angle values); and

- **Analytical methods** using 3 stage triaxial testing to determine cohesion and friction angle values.

The following rock mass shear strength parameters have been assessed from core logging and laboratory test; from which design values need to be chosen.

Cohesion

Empirical Assessment - 550kPa, RMVI = 5.5

Laboratory Assessment – 350kPa, RMVI = 2251

Friction Angle

Empirical Assessment - 41°, RMVI = 5.5

Laboratory Assessment – 39°, RMVI = 515

Design Values *(Chosen on the basis of the data source with the higher level of confidence)*

Cohesion – 550kPa

Friction Angle – 41°

Defect Plane Shear Strength

The defect shear strength has been determined form the available methods as follows:

- Empirical methods, utilising the Barton-Bandis estimation of defect plane shear strength assessed from JRC estimates recorded during core logging; and
- Analytical methods using 3 stage direct shear testing to determine cohesion and friction angle values.

The following rock mass shear strength parameters have been assessed from core logging and laboratory test; from which design values need to be chosen.

Cohesion

Empirical Assessment - 85kPa, RMVI = 4.5 (JRC) to 8.5 (JCS)

Laboratory Assessment – 54kPa, RMVI = 824

Friction Angle

Empirical Assessment - 45°, RMVI = 4.5 (JRC) to 8.5 (JCS)

Laboratory Assessment – 21°, RMVI = 503

Design Values *(Chosen on the basis of the data source with the higher level of confidence)*

Cohesion – 85kPa

Friction Angle – 45°

9.2 Lithology B

Rock Mass Shear Strength

The rock mass shear strength will need to be determined form the available methods as follows:

- **Empirical methods** (i.e. core logging data assessed to determine RMR_{89} values and cohesion and friction angle values); and
- **Analytical methods** using 3 stage triaxial testing to determine cohesion and friction angle values.

The following rock mass shear strength parameters have been assessed from core logging and laboratory test; from which design values need to be chosen.

Cohesion

Empirical Assessment - 268kPa, RMVI = 2.7

Laboratory Assessment – 705kPa, RMVI = 1740

Friction Angle

Empirical Assessment - 17°, RMVI = 2.7

Laboratory Assessment – 32°, RMVI = 460

Design Values *(Chosen on the basis of the data source with the higher level of confidence)*

Cohesion – 268kPa

Friction Angle – 17°

9.3 Lithology C

Defect Plane Shear Strength

Cohesion

Empirical Assessment - 120kPa, RMVI = 111

Laboratory Assessment –22.5 kPa, RMVI = 717

Friction Angle

Empirical Assessment - 31°, RMVI = 111

Laboratory Assessment – 29.75°, RMVI = 452

Design Values *(Chosen on the basis of the data source with the higher level of confidence)*

Cohesion – 120kPa

Friction Angle – 31

9.4 Lithology D

Defect Plane Shear Strength

Cohesion

Empirical Assessment - 140.5kPa, RMVI = 50

Laboratory Assessment –26.5kPa, RMVI = 853

Friction Angle

Empirical Assessment - 34.6°, RMVI = 50

Laboratory Assessment –25.2°, RMVI = 296

Design Values *(Chosen on the basis of the data source with the higher level of confidence)*

Cohesion – 140.5kPa

Friction Angle – 34.6

10 STABILITY ANALYSIS

10.1 Combined Rock Mass and Structural Failure (Lithology A)

Planar failure mechanisms, which will involve a failure path passing through both structure and rock mass, are anticipated within the defect planes that comprises lithology A where defect planes are undercut by the slope face, or where the variably plunging structures daylight in the slope face.

The following points list the methodology applied within the modelling package Slide to conduct this analysis:

- The Mohr-Coulomb failure criteria was used;
- Material properties were input statistically, assuming a truncated normal distribution, as discussed previously;
- A Monte-Carlo sampling method, encompassing 50,000 iterations, was utilised (and the model convergence was monitored);
- The analysis methods conducted consisted of an optimised block defined slip surface (using the non-circular function, with toe and crest projection angles of 95°-260° and 10°-85° respectively), which was determined using the Spencer method. (The analysis results are presented in Section 3.8 of this report); and
- The analysis has been conducted assuming hydrological conditions as summarised previously.
- The bedding is dipping at approximately 35° to 40°, towards the North; refer to stereographic plots (below) obtained from structural measurements undertaken during core logging.

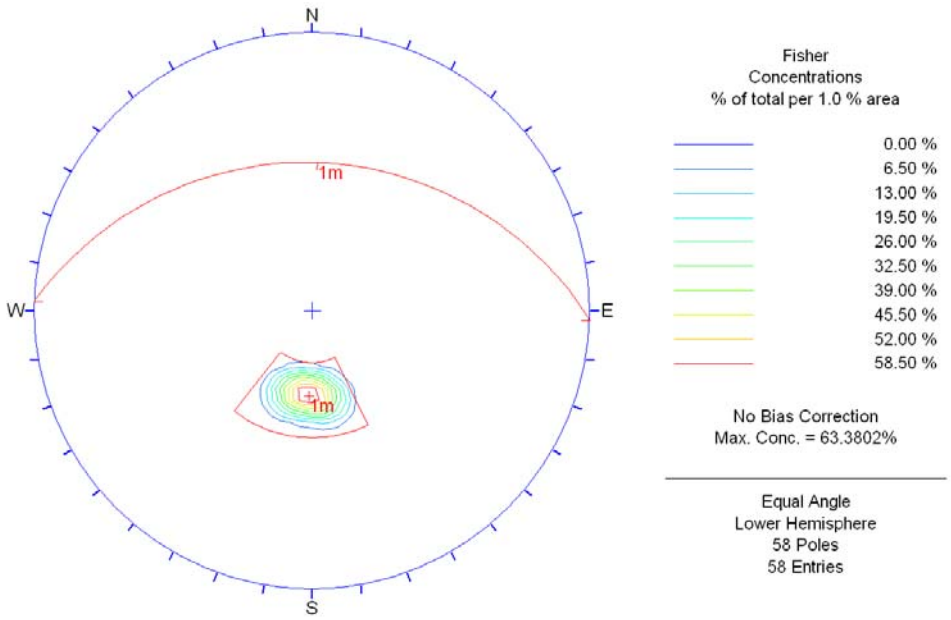


Figure 10.1.1 – Fisher Concentration of Defect Plane Orientations for Lithology A

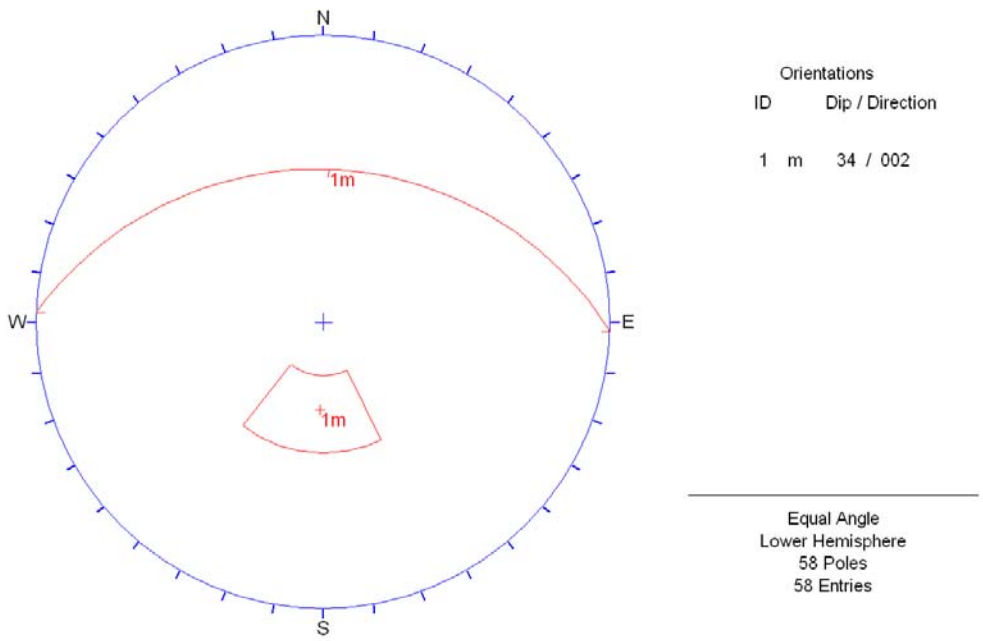


Figure 10.1.2 – Set Orientation for Defects in Lithology A

10.2 Intact Material Failure (Lithology B)

Intact material failure is anticipated within the rock mass which comprises the lithological unit B. The following points list the methodology applied by the author within the Rocscience modelling package Slide to conduct this analysis:

- The Mohr-Coulomb failure criteria was used;
- Material properties were input statistically, assuming a truncated normal distribution, as discussed previously in the paper;
- A Monte-Carlo sampling method, encompassing 1,000 iterations, was utilised (and the model convergence was monitored);
- The analysis methods conducted consisted of assuming a circular slip surface, determined using the Bishop and GLE Morgenstern-Price methods; and
- The analysis has been conducted assuming hydrological conditions as summarised previously.

10.3 Lithology C and D (Structurally Controlled Failure, Planar Block Sliding)

Structurally controlled failure mechanisms has been interpreted for lithologies C and D. Whereby planar sliding failure mechanisms, which will involve a failure path passing solely through structure is anticipated within lithologies C and D.

The following points list the methodology applied within the modelling package RocPlane to conduct this analysis:

- The Mohr-Coulomb failure criteria was used;
- Material properties were input statistically, assuming a truncated normal distribution, as discussed previously;
- A Monte-Carlo sampling method, encompassing 50,000 iterations, was utilised (and the model convergence was monitored);

- The structural planes modelled within RocPlane were determined from stereographic projections of all recorded structures, refer to figures 10.3.1 and 10.3.2 below.

Structure Orientations

Lithology C

Based on the structural data shown below in the lower hemispherical stereographic projection, it can be seen that the most critical slope face would be orientated in a south westerly direction. Hence this slope face has been modelled in detail using Rocplane.

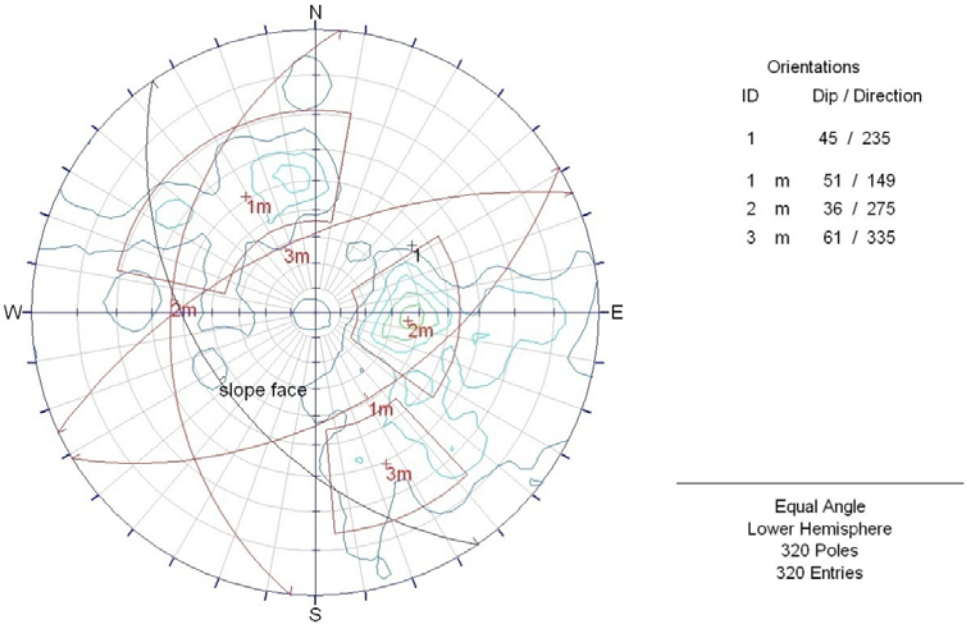


Figure 10.3.1 Stereonet Plot of Lithology C Structures

Lithology D

Based on the structural data shown below in the lower hemispherical stereographic projection, it can be seen that the most critical slope face would be orientated in a north easterly direction. Hence this slope face has been modelled in detail using Rocplane.

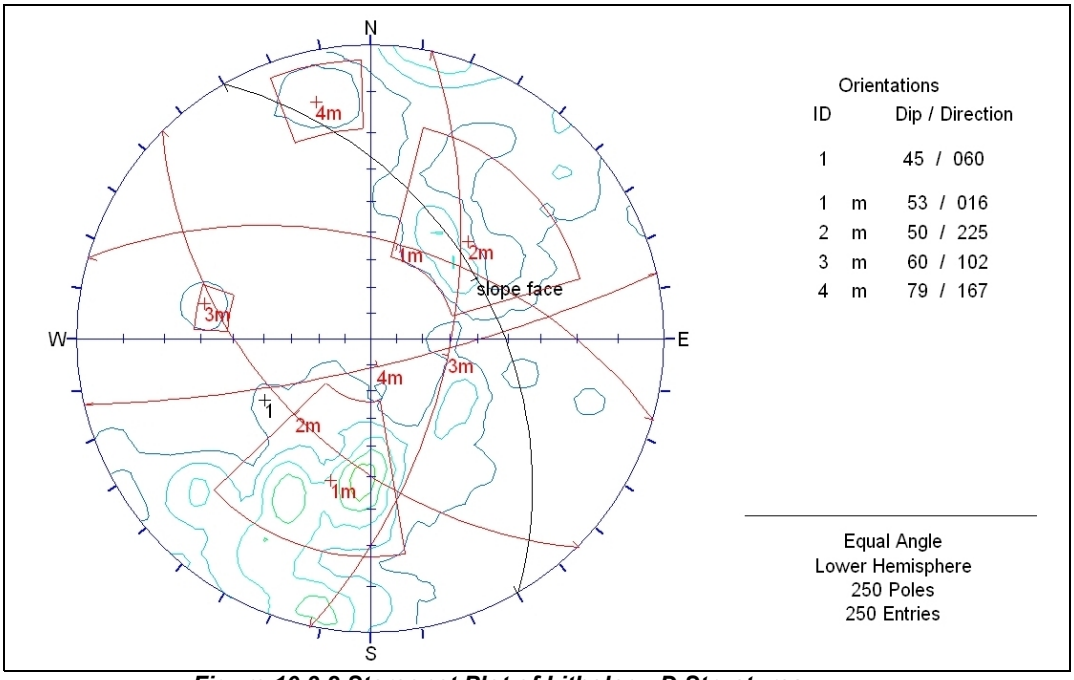


Figure 10.3.2 Stereonet Plot of Lithology D Structures

10.4 Results of Analysis

Tabulated below are the results of the stability analysis based on the design material strengths determined using the RMVI approach on a nominal slope geometry consisting of:

- A 280m high slope
- An insitu ground water table depicting partially depressurised conditions, for lithology A. There is no ground water table for Lithology B.
- The modelling was undertaking on the Rocscience software package SLIDE, the anisotropic shear strength function was utilised to effectively model the bedding planes typically with weaker infill material.
- A non circular (block defined) failure path has been presupposed due to the anticipated failure mechanism, i.e. block sliding as opposed to circular failure, due to the presence of the bedding planes within Lithology A. However for Lithology B a circular failure mechanism was modelled.

- For Lithologies C and D a structurally controlled planar block sliding mechanism was modelled in RocPlane.

- Note that the modelling process was conducted iteratively whereby numerous run was undertaken till a Factor of Safety (FOS) of 1.2 was achieved. A FOS of 1.2 is the design value for open pit slope in Western Australia. The relevant models are presented below, with the remainder presented in the appendix.

Lithology A

To achieve a Factor of Safety of 1.2, an approximate slope angle of 50° is required, assuming the design values as determined using the RMVI methodology.

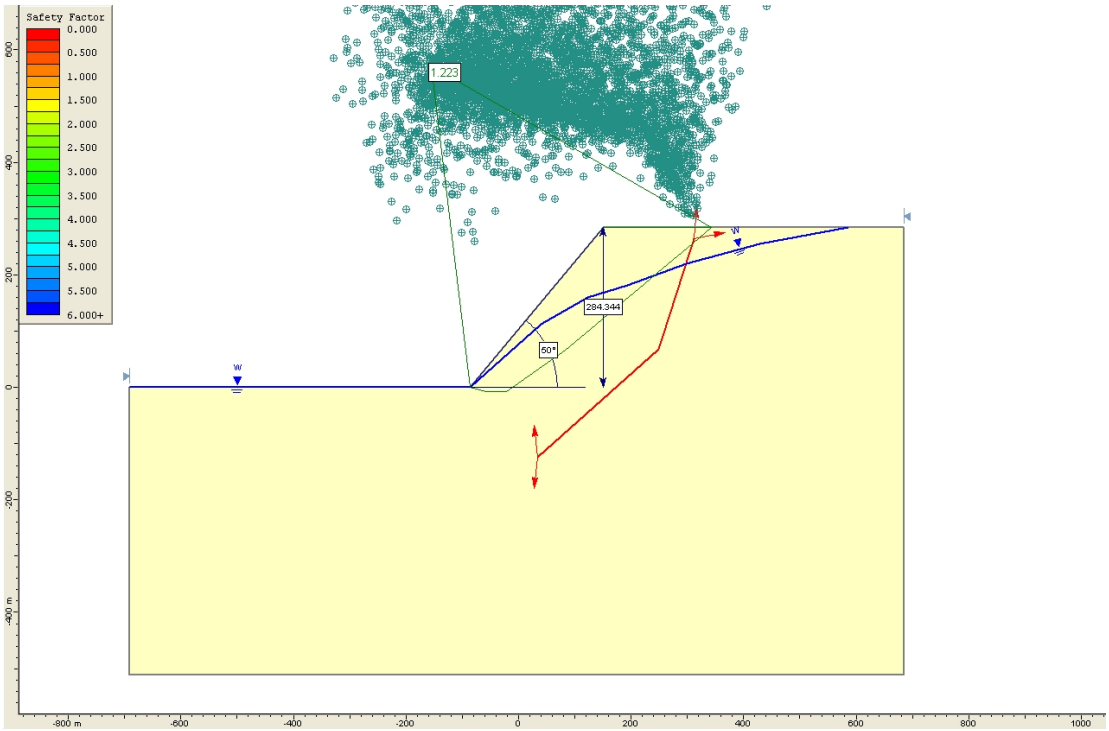


Figure 10.4.1 – Stability Modelling Results for Lithology A

Lithology B

To achieve a Factor of Safety of 1.2, an approximate slope angle of 25° is required, assuming the design values as determined using the RMVI methodology.

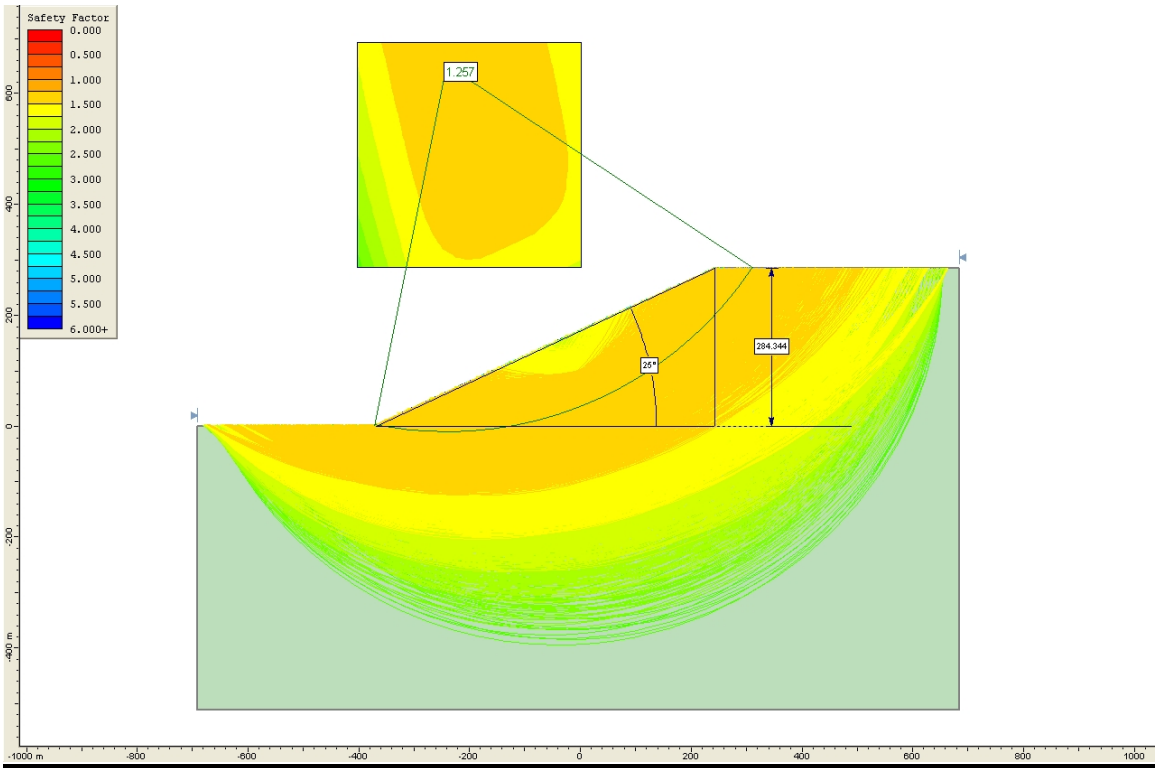
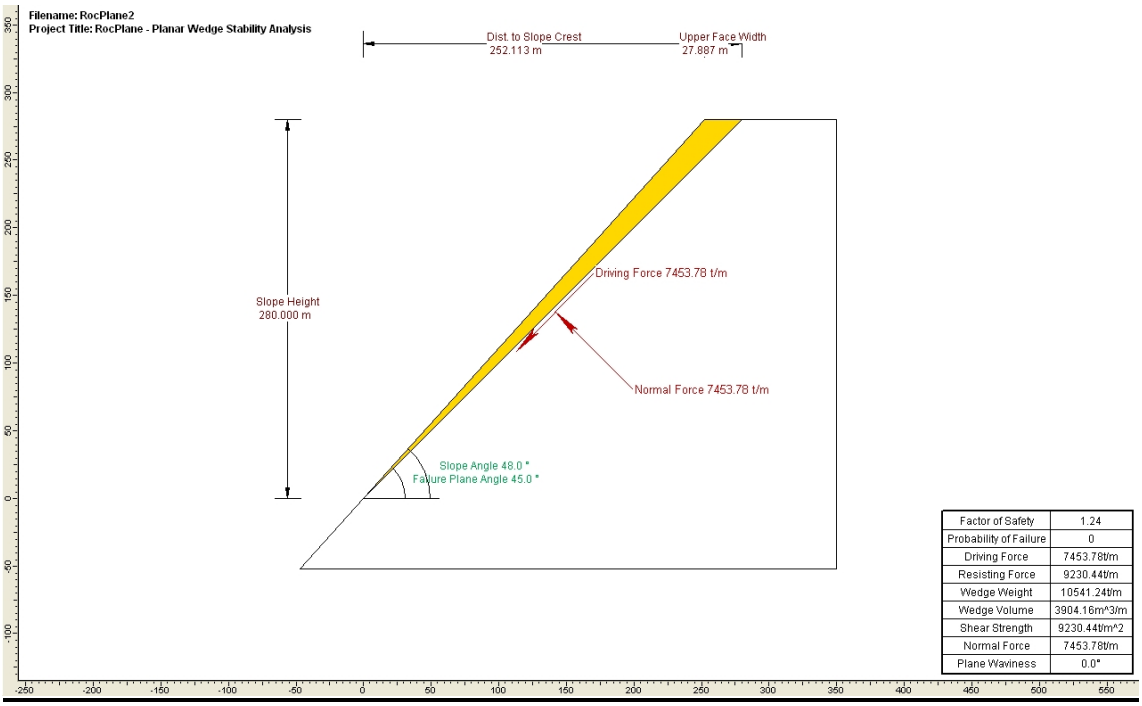


Figure 10.4.2 – Stability Modelling Results for Lithology B

Lithology C

To achieve a Factor of Safety of 1.2, an approximate slope angle of 48° is required, assuming the design values as determined using the RMVI methodology.



Factor of Safety	1.24
Probability of Failure	0
Driving Force	7453.78t/m
Resisting Force	9230.44t/m
Wedge Weight	10541.24t/m
Wedge Volume	3904.16m^3/m
Shear Strength	9230.44t/m^2
Normal Force	7453.78t/m
Plane Waviness	0.0°

Figure 10.4.3 - Stability Modelling Results for Lithology C

Lithology D

To achieve a Factor of Safety of 1.2, an approximate slope angle of 49° is required, assuming the design values as determined using the RMVI methodology.

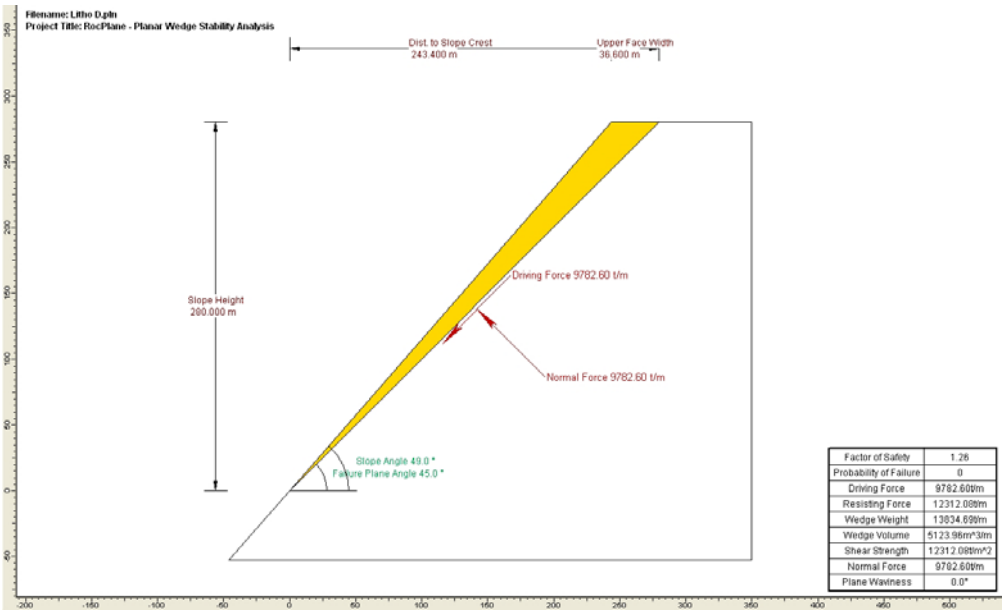


Figure 10.4.4 - Stability Modelling Results for Lithology D

11 SENSITIVITY ANALYSIS ON STABILITY MODELLING

If the traditional approach to material parameter selection were utilised, which involves choosing values based solely on laboratory testing; different slope geometries would have been obtained.

As mentioned previously, the laboratory results typically depict an upper bound estimate of material strengths for weak rock, due to the fact that majority of the samples with low strength material strengths get damaged during handling or transportation. Where strong rocks are encountered the tendency to underestimate rock strengths may occur.

Hence by using inappropriate estimate of material strengths the calculated slope geometry may not be correct, the subsections below depict this.

11.1 Lithology A

Based on the results of the sensitivity analyses undertaken on Lithology A, it can be seen that the use of the RMVI approach has facilitated the optimisation of slope angles from 26° based on the traditional approach to slope design and selection of material properties to 50° based on the utilisation of the RMVI approach.

For a typical operating mine, this results in a revenue increase of approximately $100,000 per 100m of strike length.

RMVI	Slope Angle	Reliability Coefficient	Increase in Slope Angle Based on Higher Confidence Results	Cost of Waste Movement Per Tonne	Reduction in Waste Movement Based on Optimised Geometry Per 100m	Total Cost of Waste Movement Per 100m	Anticipated Increase in Revenue Per 100m of Strike
4.5	50	0.99					
503	26	0.05	24	$3.00	33913.72	$101,741.15	$100,723.74

Table 11.1.1 – Results of Sensitivity Analyses for Lithology A

11.2 Lithology B

Based on the results of the sensitivity analyses undertaken on Lithology B, it can be seen that the use of the RMVI approach has facilitated a safer construction in this material which is generally highly weathered and weak.

The use of the traditional approach would have resulted in optimistic slope geometry which would not have been achievable, and would have resulted in an increased probability of slope failure.

RMVI	Slope Angle	Potential Failure Geometry	Probability of Failure % (Based on RMVI)	Failure Volume	Reliability Coefficient	Anticipated Failure Volume {(POF x Failure Geometry)/Reliability Coefficient)
2.67	25	59285	14.8	8774.18	0.99	8,862m3
463	70	12486	27.7	3458.622	0.05	69,172m3

Table 11.2.1 – Results of Sensitivity Analyses for Lithology B

11.3 Lithology C

Based on the results of the sensitivity analyses undertaken on Lithology C, it can be seen that the use of the RMVI approach has facilitated the optimisation of slope angles from 45.5° based on the traditional approach to slope design and selection of material properties to 48° based on the utilisation of the RMVI approach.

For a typical operating mine, this results in a revenue increase of approximately $2,500 per 100m of strike length.

RMVI	Slope Angle	Reliability Coefficient	Increase in Slope Angle Based on Higher Confidence Results	Cost of Waste Movement Per Tonne	Reduction in Waste Movement Based on Optimised Geometry Per 100m	Total Cost of Waste Movement Per 100m	Anticipated Increase in Revenue Per 100m of Strike
111	48	0.35					
452	45.5	0.05	2.5	$3.00	2304.21	$6,912.63	$2,419.42

Table 11.3.1 – Results of Sensitivity Analyses for Lithology C

11.4 Lithology D

Based on the results of the sensitivity analyses undertaken on Lithology D, it can be seen that the use of the RMVI approach has facilitated the optimisation of slope angles from 45.5° based on the traditional approach to slope design and selection of material properties to 49° based on the utilisation of the RMVI approach.

For a typical operating mine, this results in a revenue increase of approximately $5,000 per 100m of strike length.

RMVI	Slope Angle	Reliability Coefficient	Increase in Slope Angle Based on Higher Confidence Results	Cost of Waste Movement Per Tonne	Reduction in Waste Movement Based on Optimised Geometry Per 100m	Total Cost of Waste Movement Per 100m	Anticipated Increase in Revenue Per 100m of Strike
50	49	0.5					
296	45.5	0.05	3.5	$3.00	3175.49	$9,526.48	$4,763.24

Table 11.4.1 – Results of Sensitivity Analyses for Lithology D

11.5 Conclusions from Sensitivity Analysis

As presupposed, the author has demonstrated via the use of the four case studies that:

a. Where the traditional approach to material property selection and slope stability modelling is utilised; there is a tendency to underestimate rock strengths and hence slope geometry, when dealing with strong rocks. This would invariably result in conservative slope designs, which would result in higher strip ratios for operating mines, adding to operating costs.

b. Where the traditional approach to material property selection and slope stability modelling is utilised; there is a tendency to overestimate rock strengths when dealing with weak materials. This would hence result in somewhat optimistic slope designs which would not be achievable, and result in an increased likelihood of failure.

12 CONCLUSIONS

12.1 Calibration of rock mass variability index with statistical parameters

As mentioned previously within Section 3.1, the author has attempted to calibrate i.e. determine a relationship between the coefficient of variability and the Rock Mass Variability Index. This relationship has been derived using the empirical and analytical data presented above as part of case studies A, B, C and D.

It is the authors intention that the derived relationships presented below can be used by rock mechanics practitioners in the following comportment when assessing rock mass conditions for the purpose of pit slope stability modelling:

- To assist in the selection of the number of samples to subject to laboratory strength measurements.
- Given a particular degree of material variability the use of the RMVI methodology would assist in selecting appropriate material properties for stability modelling, specifically where:
    a. The materials are relatively strong and there exists the possibility to underestimate material strength in solely relying on laboratory results as demonstrated in Case Studies A, C and D; or
    b. Where the materials are weak there exists the possibility to overestimate material strengths when relying solely on laboratory testing results as observed in Case Study B.

There are some points in relation to the application of the Rock Mass Variability Index for the purposes of material property selection and rock mass stability modelling that the author would like to highlight.

- The relationships developed by the author relating the Rock Mass Variability Index to coefficients of variability, are primarily applicable to rock masses with

Rock Mass Ratings (RMR_{89}) after Bieniawski (1989) of between 30 and 45, indicative of poor to moderately competent rock.

- The author cautions the reader from applying these relationships to for RMR_{89} values greater than 50, as they may not be completely accurate.
- As part of the final conclusions on this research work the author is proposing that others undertake an extensive data analysis (empirical and analytical) of available rock types over a range of RMR_{89} values. Ideally this should cover RMR_{89} values of between 20 and 80.
- It should be also noted that the charts and derived relations ships presented below are biased towards sedimentary and foliated rock types, there is a possibility that they may have to be modified somewhat when being applied to say igneous or metamorphic rock types that are not heavily foliated within the same RMR_{89} range.

12.2 Discussions of Results

Figure 10.1.1 represents the relationship the author has derived between the Rock Mass Variability Index, the Coefficient of Variability and the number of samples for the data sets presented as part of case studies A, B, C and D.

Based on the information in this chart the author has derived the following relationships for future application when assessing rock mass parameters for rock types with RMR_{89} values between the range of 30 and 45.

1) There appears to be too much scatter in the data set, to observe any definite relationships between the Coefficient of Variability and the number of samples.
2) A very good correlation was noted between the Rock Mass Variability Index and the number of samples for a particular data set. The following relationship has

been derived by the author, with a correlation coefficient of 0.86, signifying a strong trend. Where $\boldsymbol{RMVI = 3862 \times N^{-0.88}}$, where N is the number of samples.

3) For Rock Mass Variability Indices values less than 50, the following relationships relating Rock Mass Variability Index and Coefficient of Variability have been derived by the author for the specified intervals:

   i. $COV = 12.93 \times RMVI$ for $RMVI < 5$

   ii. $COV = 12.3 \times RMVI$ for $5 < RMVI < 10$

   iii. $COV = 5.78 \times RMVI$ for $10 < RMVI < 15$

   iv. $COV = 1.4 \times RMVI$ for $25 < RMVI < 50$

4) For Rock Mass Variability Index values greater than 50, the following relationship relating Rock Mass Variability Index and Coefficient of Variability has been derived by the author; $\boldsymbol{COV = 32.6 \times e^{0.0006.RMVI}}$, (For RMVI>50).

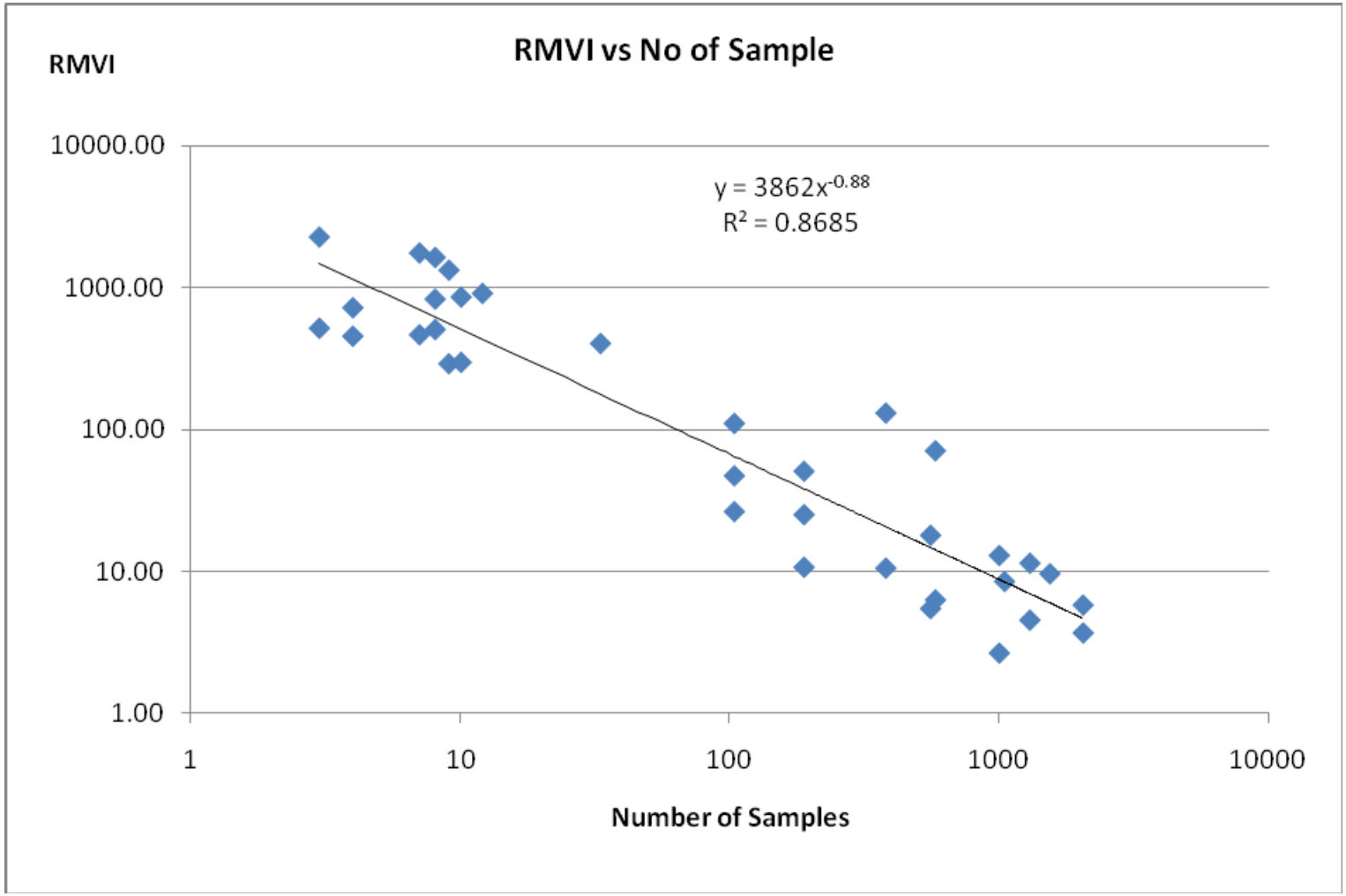


Figure 12.2.1 – Relationship Between RMVI and Number of Samples

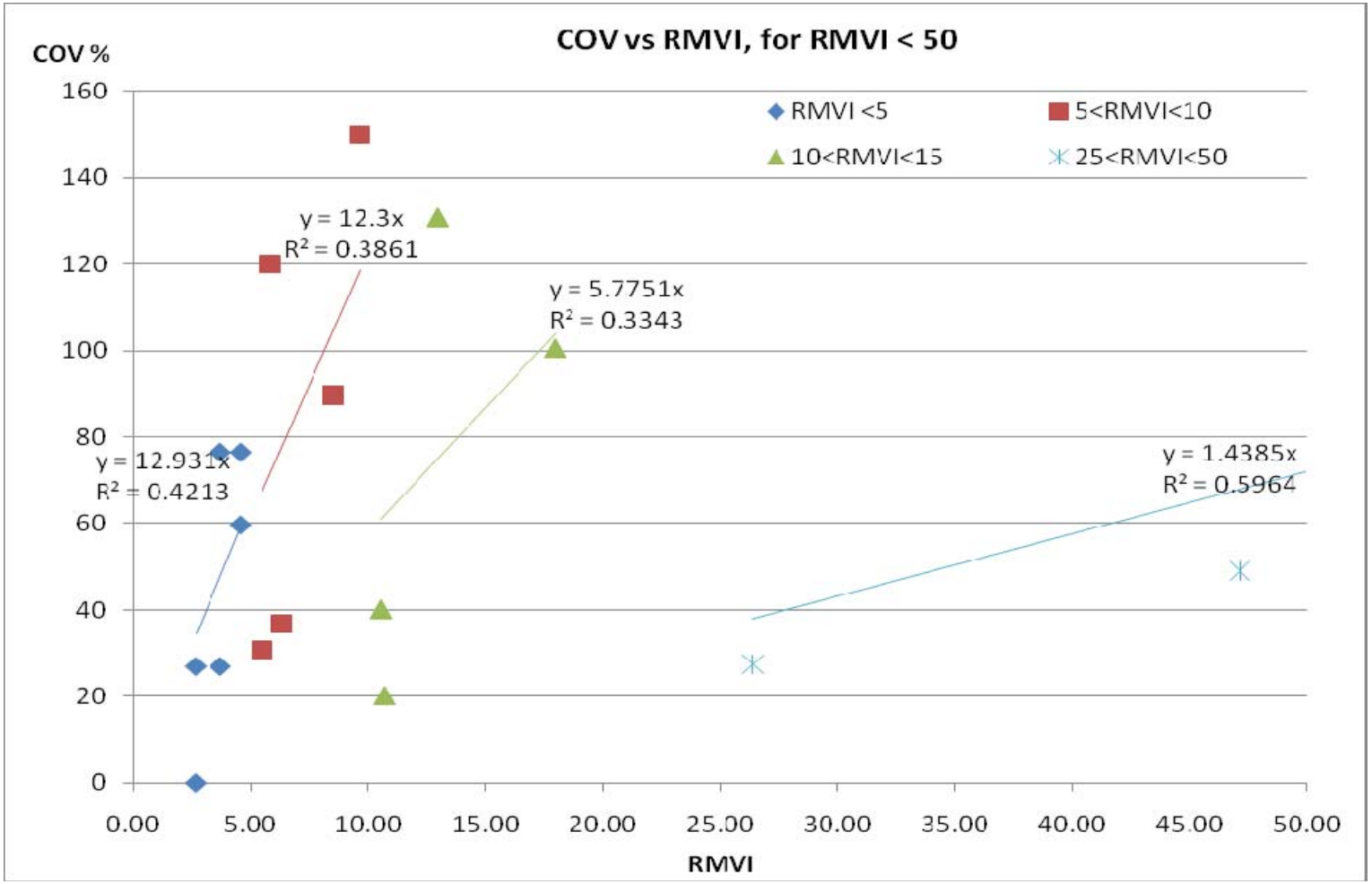


Figure 12.2.2 – Relationship Between RMVI, COV and Number of Samples for RMVI <50

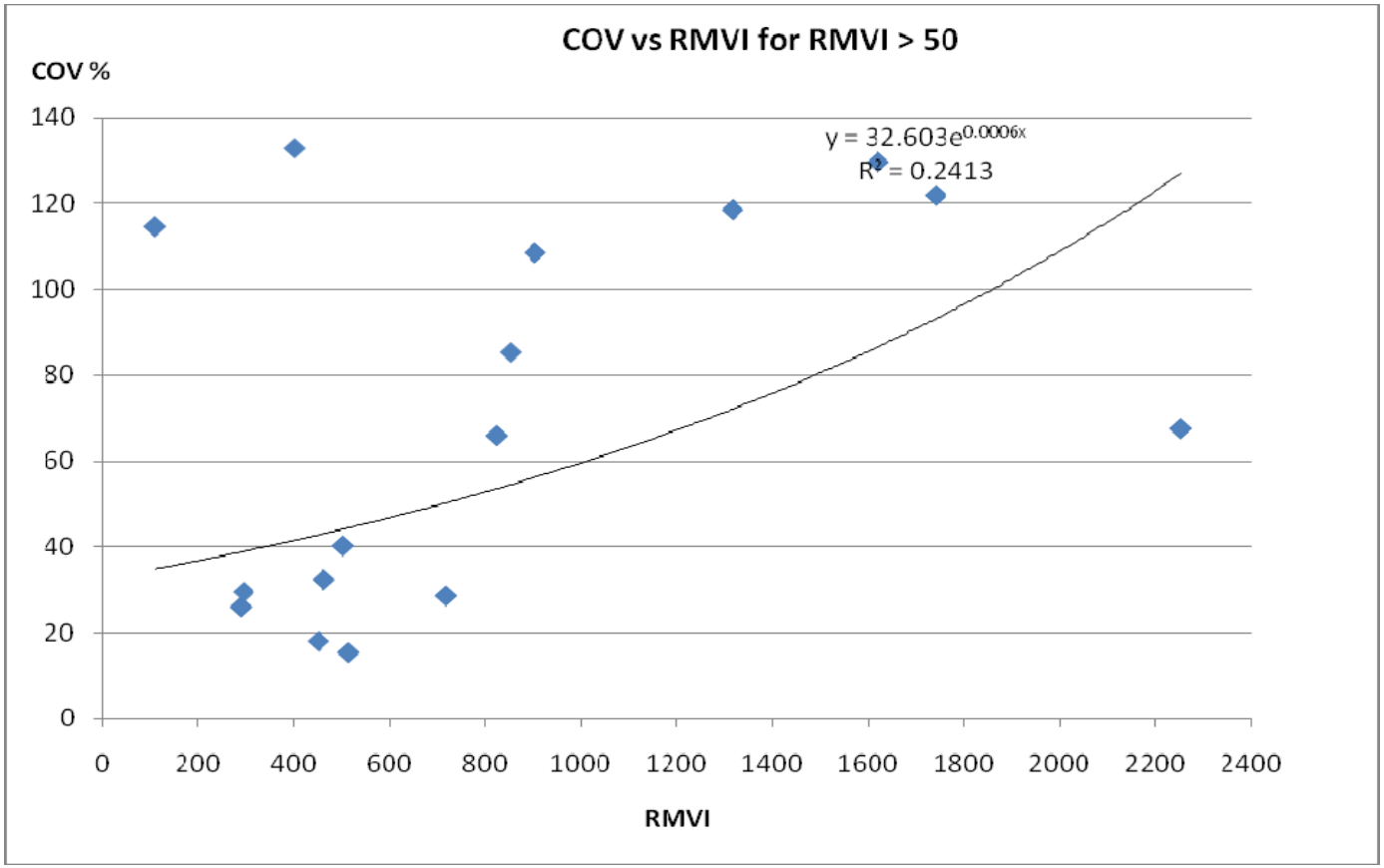


Figure 12.2.3 – Relationship Between RMVI, COV and Number of Samples for RMVI >50

12.3 Proposed Application Methodology of the Rock Mass Variability Index for Design Purposes and Further Work

It is suggested that when applying the Rock Mass Variability Index for undertaking geotechnical data collection for rock mass and pit slope design purposes, it should only be applied to rock masses that have a RMR_{89} rating of between 30 and 40.

Suggested methodology for application of Rock Mass Variability Index:

I. The practitioner should be able to make an assessment of the required confidence of the requirement geotechnical assessment. If it were 85 percent, a Rock Mass Variability Index of say 5 would be chosen.

RMVI	Reliability Coefficient	Comment
<3	0.99	Extremely Reliable Data Set; Low Insitu Variability & Significant Number of Samples
3 to 7.5	0.85	Very Reliable Data Set; Some Insitu Variability & Significant Number of Samples
7.5 to 20	0.65	Reliable Data Set; Insitu Variability Noted & Significant Number of Samples
20 to 50	0.50	Moderately Unreliable Data Set; Significant Insitu Variability Noted & Inadequate Number of Samples
50 to 100	0.35	Very Unreliable Data Set; Significant Insitu Variability Noted & Inadequate Number of Samples
100 to 150	0.15	Extremely Unreliable Data Set; Significant Insitu Variability Noted & Inadequate Number of Samples
>150	0.05	Inadequate data to determine statistical validity

II. Using the following relationship $\boldsymbol{RMVI = 3862 \times N^{-0.88}}$ the practitioner would evaluate the number of samples required, i.e. in this case 1912. Obviously it would be unfeasible to obtain all these samples from laboratory testing due to the cost implications. However this would provide an indication of the required data points which could be relatively cheaply obtained from empirical means from say drill core assessments.

III. Once the required number of data points are collected, the true variability of the material can be assessed. Using the relationship ***COV = 12.93 x RMVI,*** *(For RMVI<5)*, the expected coefficient of variability of the material should be in the order of 60 percent.

IV. Hence when collecting data for the respective components of the particular rock mass the practitioner could expect about a 60 percent degree of variability, in say the triaxial, direct shear and unconfined compressive strength data. Should the calculated coefficients of variability not be in this order, it is probable that the further sampling is required for verification purposes.

V. As mentioned previously this work / research has focussed on the rock mass as a whole, rather than taking into account discrete structural components that comprise the overall rock mass. There is further scope to investigate the application of the RMVI methodology to the individual structural components to investigate the structural geological variability and the effects on overall estimation of rock mass characterisation.

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