CURRENT PRACTICES FOR ESTIMATION OF STRENGTH AND DEFORMATION PROPERTIES OF WEAK ROCK MASSES FOR GEOTECHNICAL APPLICATIONS

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Signed on 30 May 2018 in Johannesburg

DECLARATION

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

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(Signature of Candidate)

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ABSTRACT

A weak rock mass comprises a collection of material with diverse characteristics and there is thus no single description for weak rock masses. This report summarises developments made in the understanding of weak rock mass, based on measurable parameters.

Available tests predominantly measure the compressional strength of intact rock material. The shear strength is then estimated through existing failure criteria, since it is very difficult to obtain the shear strength of rock directly. Wiid (1981) offered an alternative testing technique, ideal for the measurement of shear strength of very soft to soft rock, in the form of a modified vane shear test and this technique is explored further in this report.

Additionally, current modelling practices for rock masses generally consider shear strength criteria. However, unexpected failures in major excavations indicate the importance of damage mechanics and the presence of tensile strains in the rock (mass). Through correlations between measurable parameters, a conceptual model for rock strength, is suggested.

To my loving parents

Johannes de Flamingh Odendaal and Mary Anne Odendaal.

“I am more and more amazed about the blind optimism with which the younger generation invades this field, without paying attention to the inevitable uncertainties in the data on which their theoretical reasoning is based and without making serious attempts to evaluate the resulting errors”

- Karl Terzaghi

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GLOSSARY

2D	Two Dimensional
3D	Three Dimensional
CSW	Continuous Surface Wave
E	Young's modulus
Ei	Young's modulus for rock material
E_{RM}	Deformation modulus for rock mass
G	Shear modulus
GSI	Geological Strength Index
ISRM	International Society for Rock Mechanics
JRC	Joint Roughness Coefficient
m_i	Material constant in Hoek Brown criterion
MRMR	Mine Rock Mass Rating
P-Wave	Compression Wave
Q	Description of the rock mass stability of an underground opening
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SAICE	The South African Institution Of Civil Engineering
SAIEG	South African Institute for Engineering and Environmental Geologists
SPT	Standard Penetration Test
S-Wave	Shear Wave
UCM	Uniaxial Compression Modulus
UCS	Uniaxial Compressive Strength
UCS_i	Uniaxial Compressive Strength of rock material
UCS_{RM}	Uniaxial Compressive Strength of rock mass
ν	Poisson's Ratio
σ_1	Major Principal stress
$\sigma_1 - \sigma_3$	deviatoric stress
σ_3	Minor Principal stress
σ_t	Tensile strength
τ	Shear strength
ε	Rock strain
I_s	Point load strength

CHAPTER 1 - INTRODUCTION

This report covers developments made in the understanding of weak rock masses. The scope and objectives for this study are described within the following sections.

1.1 General Introduction

For any construction in or on a rock mass, an appropriate geotechnical design requires a reasonable estimation of the strength of the rock (mass), as well as the material's resistance to deformation under a specified load. This will enable the engineer to design suitable ground improvement and / or support for the construction. Up until the 1980's, soft rock was generally considered a "fringe" of either soil or rock mechanics, depending on the background of the engineer working with the material. This led to significant errors in the estimation of weak rock properties, and to undue conservatism.

Within available studies on soft rock, researchers now recognise that both compressibility and associated pore water pressures, as well as fracture initiation and network characteristics are significant. Thus, soft rock embodies concepts considered in the fields of both soil and hard rock mechanics (Johnston, 1991).

In this report, a detailed literature review will aim to summarise developments made in the understanding of the behaviour of soft rock and its weak rock mass equivalent. By understanding weak rock masses, confidence in designs within these materials will increase, and this will ultimately lead to improved designs.

The focus of the research being carried out for this report is limited to the strength and stiffness of continuous weak rock masses under pre-failure conditions.

A weak rock mass embodies a collection of material with adverse characteristics. The material considered within this study (rationalised by Santi, 2006, Nickmann et al, 2006 and Kanji, 2014) is illustrated in Figure 1.

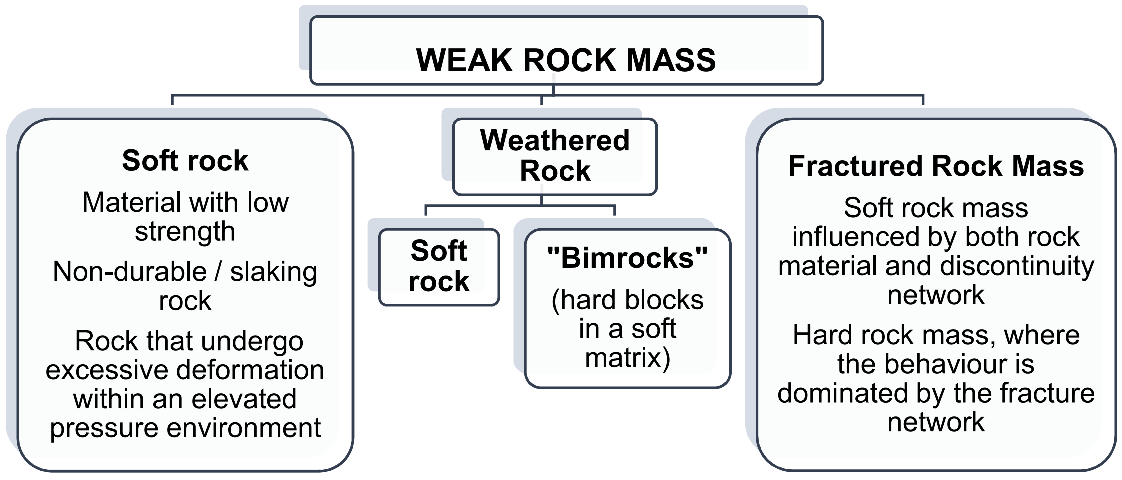


Figure 1: A weak rock mass assemblage

Geotechnical designs are based on the anticipated performance of the geo-material within the proposed design. Predictions of the performance are made by means of available empirical design charts or numerical modelling.

With the major developments that have taken place in technology, computer models have become highly sophisticated and generally add to the degree of confidence in the predictions. These models are, however, still a function of the constitutive relationships from theoretical and empirical failure envelopes, estimated with parameters obtained as either a minimum or an average value from test results of field and laboratory investigations (adapted from Johnston (1991) and Barton (1999)).

Prior to these technological advances, a reasonable estimate of measurable properties was considered to an accuracy within 10 – 20%, with a general opinion that the function of the rock engineer is not to compute accurately, but judge soundly (Bieniawski, 1974). With the shift in focus from experience and engineering judgement to accurate behavioural analysis, through analytical techniques, an understanding of what the parameters portray, and their effect on the assumptions made within the model, is essential to obtain sound predictions.

The properties may be estimated, either by:

- Measured index properties, supplemented by an existing theoretical or empirical scheme (like a strength criterion) to give an estimation of the parameter within specified stress conditions; or

- Specialised test measurements of parameters representative of the desired stress state. Available specialised testing techniques include both small scale laboratory tests and large-scale tests within the field.

A summary of available testing methods is given in Figure 2.

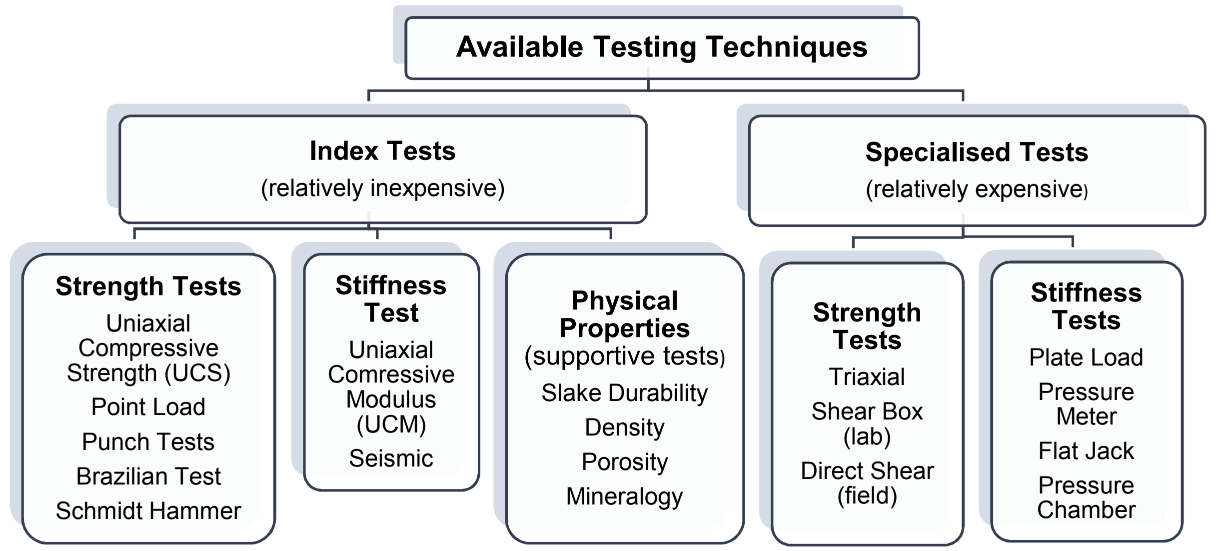


Figure 2: Testing Techniques

In the literature review, each of the diverse rock characteristics, with available testing techniques to define these features, will be considered.

1.2 Study Outline

The literature review will look at each of the subdivisions illustrated in Figure 1 as individual sections, viz:

- Section 1 – Soft Rock;
- Section 2 – Fractured rock masses;
- Section 3 – Bimrocks.

In each section, consideration will be given to the rock (mass) strength and stiffness estimation based on measurable parameters.

In Section 4 a summary will be presented of available testing methods, indicating for each the advantages, limitations and applicability for soft rock (masses).

Section 5 is dedicated to current analysis techniques used to predict the rock (mass) characteristics for a given design.

The literature review is followed by a chapter on experimental work. This section presents the results of a limited testing program for the evaluation of the potential of a non-standard vane test method to measure the shear strength of soft rock.

Finally, the correlations between the measured parameters are explored and a conceptual model to define rock strength is proposed.

1.3 Research Objectives

This report is aimed at exploring the general understanding of weak rock masses, by:

- Describing the prominent characteristics of a weak rock (mass);
- Reviewing current tools available for the estimation of strength and stiffness parameters of weak rock (masses);
- Exploring general trends and/or correlations which best reflect the strength and stiffness of weak rock (masses).

CHAPTER 2 - LITERATURE REVIEW

Dealing with weak rock masses, one will encounter a combination of undesirable behaviours, including, but not limited to, low strength, high plasticity, and significant weathering rates (and associated transformed characteristics). The material behaviour is modelled using strength and stiffness estimates that are based on measurable parameters. Additionally, unlike other engineering disciplines, these estimates are not fixed constant values, but are characterised by variability that is dependent on the surrounding environment, such as confining pressure and groundwater conditions.

The following sections explore developments made in the field of weak rock (masses).

2.1 Soft Rock

Traditionally, the term 'soft rock' was used to describe a rock with low strength. Subsequently, the selection grew to include rocks with high clay content or other unstable minerals, weathered rock and rock (masses) subjected to high pressures and/or temperatures.

2.1.1 *Soft rock due to low strength*

The evaluation of rock with low strength is primarily made by means of the Uniaxial Compressive Strength (UCS) test, which is one of the most commonly used index tests in providing design data.

Various researchers have tried to specify the strength range covering soft rock. Here, arbitrary values were selected based on arguments such as the strength of concrete (for the upper limit), and refusal of the Standard Penetration Test (SPT) spoon (for the lower limit). Some of the documented ranges include:

- 0.25 - 25MPa (ISRM, 1981a)
- 0.50 - 25 MPa (Johnston, 1991)
- 1.00 - 20MPa (Santi, 2006)
- <20MPa (Agustawijaya, 2007)

<u>UCS Test</u>

A UCS value comprises of a rock strength estimate derived from a core sample with a reference diameter of 50mm and specified length, that is compressed under atmospheric pressures up to failure. A minimum length is required to eliminate elevated strength measurements due to bedding errors.

The ISRM (1981b) suggested a ratio of $2.0_{(length)}:1_{(diameter)}$ to $2.5_{(length)}:1_{(diameter)}$ core sample. However, a study by Agustawijaya (2001) showed reasonable strength estimates for soft rock samples with a ratio of $1.6_{(length)}:1_{(diameter)}$.

This simple laboratory test provides a single value that, if not considered carefully, may be misleading. Additional factors that should be taken into consideration for the UCS interpretation and application for rock strength estimates include:

1. *Mode of failure*

   Within the literature, UCS values are often presented without any reference to the failure mode. This can be either axial splitting (sometimes referred to as axial cleavage), with associated lower strength estimates dependent on the rock's tensile strength, or shearing with associated higher strength values (Johnston, 1991).

2. *Scale*

   It is well known that when sample sizes increase, lower strengths are measured. This relationship is exponential and occurs up to sample sizes of at least one metre (Douglas, 2002). This is illustrated in experimental work by Hoek and Brown (1980) and presented in Figure 3.

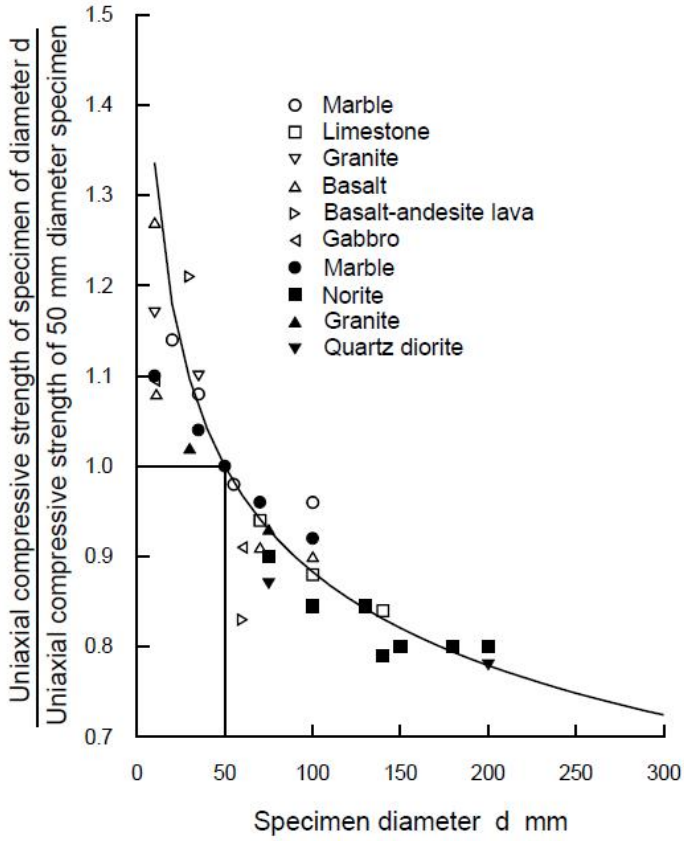


Figure 3: Effect of sample size on strength estimations (Hoek, 2007)

Recorded strength reduction estimates between laboratory (including UCS estimations) and field tests include:

- Up to a factor of 10; (Barton, 1976)
- Up to 30%. (Hoek, 2000)

3. Degree of sample saturation

Experimental test results on various rock types, including sandstone (Agustawijaya, 2007), mudstone (Jumikis, 1966) and quartzite (Colback and Wiid, 1965) confirm that intact rock material loses its strength and elastic range when wetted. However, the effect of water content appears to be more significant in sedimentary deposits compared to crystalline rock (Wong et. al., 2016).

This non-linear relationship is largely affected by the rock composition (Johnston, 1991).

Sandstone loses up to 38% of its strength at a moisture content of between 0% and 1%, with insignificant strength reduction as the moisture content is increased further (Agustawijaya, 2007).

With mudstone, there is a gradual strength reduction with an increase in the degree of saturation up to 100%. Jumikis (1966) found that the measured UCS values of saturated shale may be up to one order of magnitude lower than for a dry sample.

Tests may be carried out on either dry or saturated samples. Oven dried samples will give the upper bound and saturated samples the lower bound strength estimate.

In addition to the considerations identified above that affect all rock types, further considerations become significant for soft rocks, one of the main concerns being the testing rate.

4. Testing Rate

As mentioned in the Introduction, soft rock has a compressible skeleton. Upon loading, pore water pressure rises, and if not given enough time to dissipate, will lower the strength of the material. Within soil mechanics, this effect is well understood, and designs are based on effective strength values. This value is estimated with (Das, 2002):

$$\sigma` = \sigma - \mu \qquad \textbf{Equation 1}$$

Where

- $\sigma`$ is the effective strength;
- σ is the total strength; and
- μ is the pore water pressure.

Standard procedures for soil tests differentiate between drained and undrained testing. A test is drained when the pore water can dissipate, and the excess pore water pressure remains zero throughout the test (the sample does however remain saturated). Here, a consolidation test is carried out to determine the rate at which the sample should be compressed up to failure. At this rate, no excess water pressures will develop. An undrained test involves a closed system where the pore water is not allowed to dissipate, and the water pressure is measured throughout the test. These tests derive different strength parameters, with no common correlation between them.

However, work by Tho et al. (2011), indicated a possible correlation between effective drained and un-drained triaxial strength estimation, which is dependent on the proportion of water within the sample. The correlation equation is:

$$c'_{CD} = \frac{c'_{CU}}{1 + \left(\frac{0.01W}{n}\right)} \qquad \textbf{Equation 2}$$

Where

- c'_{CD} is the effective cohesion for consolidated drained test;
- c'_{CU} is the effective cohesion for consolidated un-drained test;
- W is the water content (%); and
- n is a constant (2.009 is recommended by Tho et al., 2001)

Within standard UCS tests, designed for rock testing, drained conditions for either saturated or dry samples are assumed with testing carried out at standardised rates (ISRM, 1981b). These rates may either be a constant rate that is slow enough for failure to occur between 5 and 15 minutes of testing or a selected rate between 0.5 to 1 MPa / sec. Although these rates are reasonable for hard rock, they will result in excess water pressures, lowering the strength estimate of soft rock (Johnston, 1991).

Considering Johnston's (1991) proposal that soft rock behaves in a similar manner to over-consolidated clay, it is considered reasonable that, for the purpose of this study, the Unconfined Compression Test on saturated clay is an acceptable analogy to explore this concern. With this special type of test, the axial load is applied rapidly to a saturated, unconfined clay sample, and the measured *undrained shear strength* is slightly lower, but still a reasonable estimation of the strength of a triaxial unconsolidated-undrained compression test (Das, 2002).

From this analogy, the tested UCS measurements are not significantly affected by the excess water pressure during testing under standard rates.

An experimental study by Chen et al. (2007), confirmed the effect of excess pore water pressure, but it is considered insignificant, compared to the increase in the UCS value with an increase in strain rate. The results from this study are shown in Figure 4.

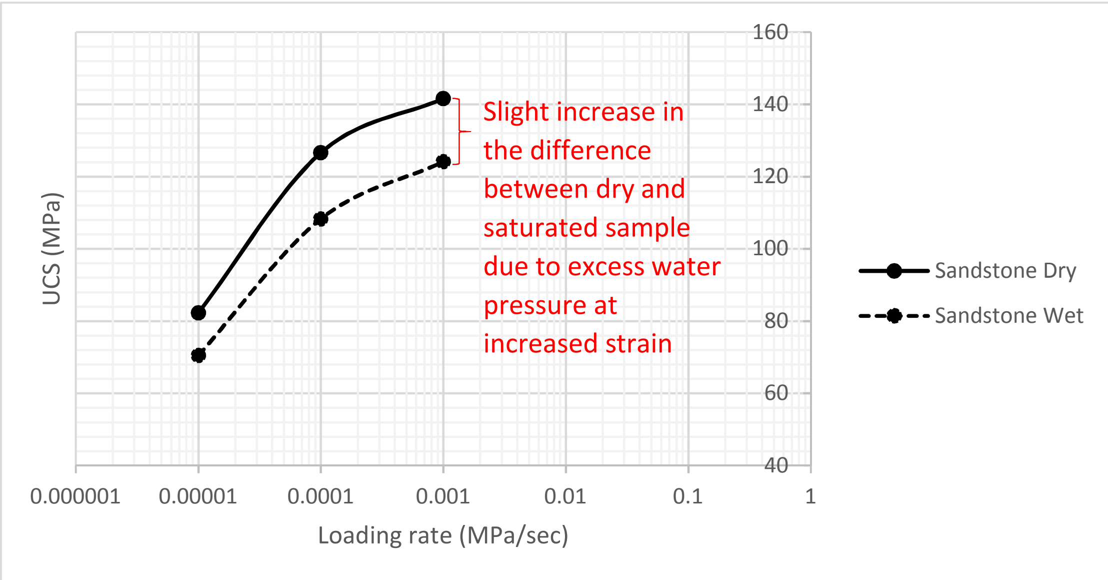


Figure 4: Effect of test rate on sandstone (Chen, et al., 2007)

An additional study by Sartkaew and Fuenkajorn (2013) on dry salt samples under varying temperature and testing conditions, showed a similar trend as illustrated in Figure 5.

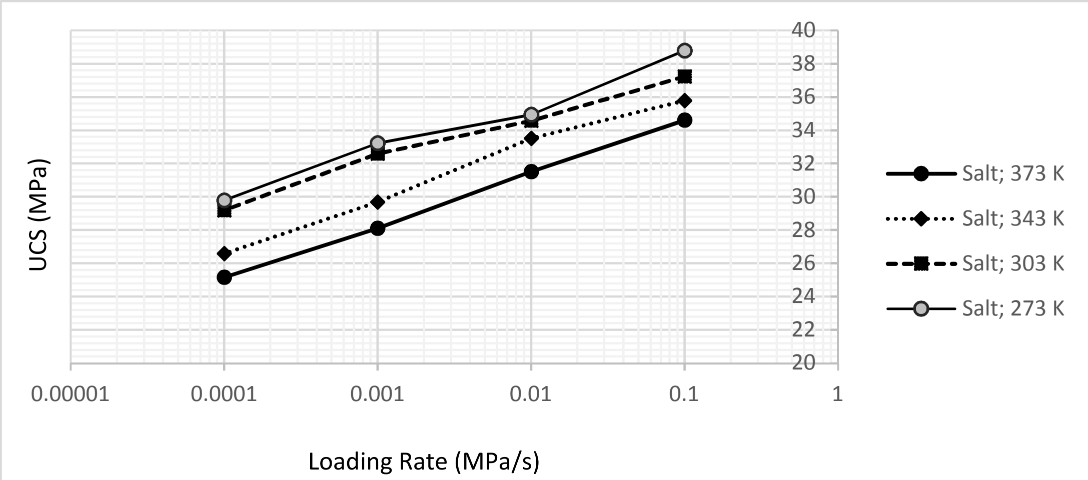


Figure 5: Correlation of salt strength under various loading rates &temperatures (Sartkaew and Fuenkajorn, 2013)

For soft rock in general, the resultant increase in strength under standard testing rates appears to be dependent on the material composition. Thus, additional work is required for a quantitative analysis for all rock types. However, the studies presented above indicate that UCS measurements on soft rock samples that is tested under standardised loading rates, are generally grossly overestimated (Chen, et al., 2007 and Sartkaew and Fuenkajorn, 2013).

Rock is known to undergo continuous deformation under constant stress over time, this is commonly known as creep. The amount of deformation is more in soft rock. There are three different phases of creep, as illustrated in Figure 6.

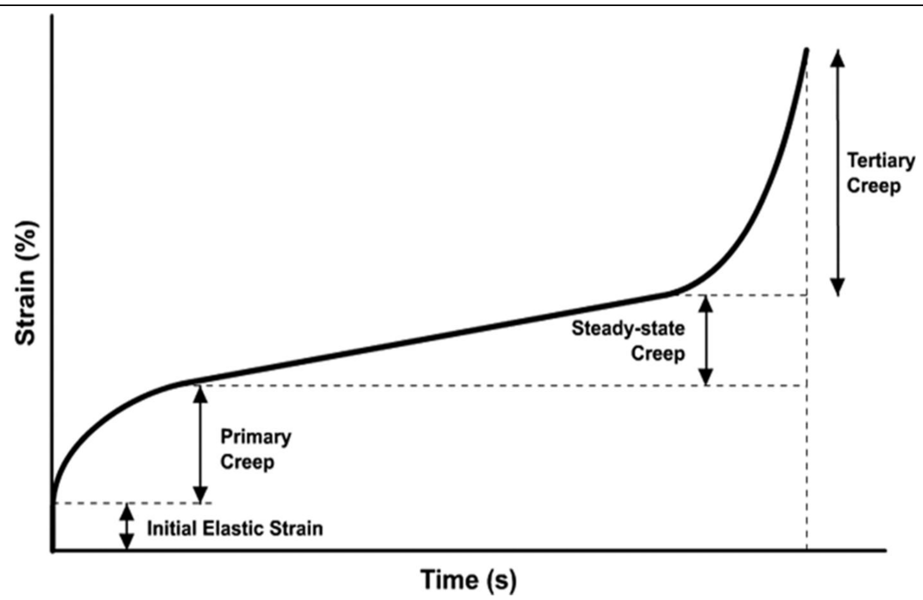


Figure 6: Creep behaviour over time (Mukherjee et al., 2016)

Tests carried out with standard loading rates do not allow the rock to deform during the test. Thus, it is my postulation that the higher UCS measurements of soft rock are related to the lack of strain softening that will occur during the instantaneous strain phase of creep in the field.

Alternatively, Chen, et al. (2007) considered the findings of Atkinson and Cook (1993) who identified a general decline in crack growth rate as loading rates increase. They considered this as a plausible explanation for the increase in strength with increasing loading rates.

At zero confinement, the cohesive strength is considered the ultimate soil strength, which is also the total shear strength. The estimated cohesive / shear strength, derived from the UCS is dependent on the testing method. The undrained shear strength of a fully saturated soil sample is independent of the confining pressure and the UCS is considered twice the undrained shear strength of the material in accordance with the equation below (Das, 2002):

$$\tau_u = \frac{UCS}{2}$$ **Equation 3**

Where

- τ_u is the Undrained Shear Strength; and
- UCS is the Uniaxial Compressive Strength.

Consideration of estimated strength of saturated drained samples

By considering the relationship between UCS and undrained cohesive strength (Equation 3) and the relationship between drained and undrained strength (Equation 2), a rough estimation for the drained cohesive strength of a saturated sample based on the UCS should be:

$$Cohesion_{(rock\ strength)} = \frac{UCS}{2.996}$$ **Equation 4**

Consideration of estimated strength of dry samples

Based on Equation 2, when the UCS of a dry soil specimen is measured, the measured cohesion is expected to be the drained strength (UCS/2).

Rock hardness / strength does not only provide an estimation of the total load the rock can sustain without failure, but also affects the stress-strain response prior to failure. Hard rock is associated with brittle failure (strain softening in Figure 7), and soft rock with ductile deformation, facilitated by failure through crushing, sliding and rotation of small intact fragments (strain hardening, or elastic-perfectly plastic behaviour in Figure 7) (Johansson, 2005).

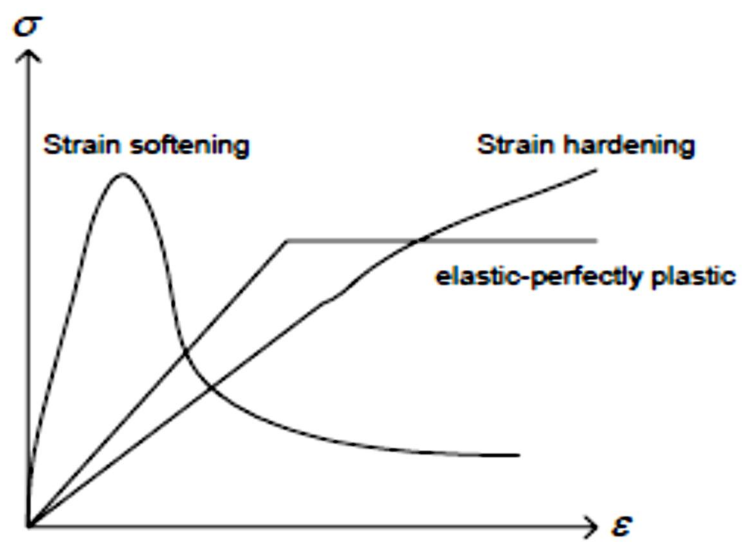


Figure 7: Stress – strain behaviour under axial loading (Johansson, 2005)

Like soft rock, soil is characterised by an elastic-perfectly plastic or strain hardening relationship with no significant strength loss at failure. Thus, it could be argued that, by association, the shear strength of soft rock may be estimated with either Equation 3 or Equation 4.

However, the rock response is also affected by confining pressures. It has been found that even soft rock displays brittle behaviour under low confinement, and hard rock may behave in a ductile manner under very high confining pressures (Johnston, 1991).

Thus, it is considered that the shear strength estimates for soft rock should rather be based on correlations derived in hard rock mechanics. Robertson (1970) provided an approximate relationship, which is considered as suitable for practical applications (Stacey and Page, 1986) as:

$$\tau_D = 0.16 \times UCS \qquad \textbf{Equation 5}$$

Where

- τ_D is the Drained Shear Strength; and
- UCS is the Uniaxial Compressive Strength.

Another difficulty faced by practitioners is the availability of samples of sufficient length to carry out a UCS test. This led to various existing correlations with indirect testing methods (e.g. point load, punch test and vane shear tests), and physical material properties (e.g. absorption).

Point Load tests

Point load tests comprise of a simple, inexpensive technique designed to test rock strength. Various researchers have suggested correlations between the average UCS values and point load strength, for example:

- UCS= 22 x Is (Hard rock) (ISRM, 1985)
- UCS= 23 x Is (Hard rock) (Hoek and Bray, 1974)
- UCS= 24 x Is (Hard rock) (Broch and Franklin, 1972)
- UCS= 24 x $Is_{(50)}$ (Bieniawski, 1974)
- UCS= (7.4 - 17.6) x Is (Sandstone) (Forster, 1983)
- UCS= (5-24) x Is (Chalk) (Bowden et al., 1998)
- UCS= 14 x Is (Soft rock) (Chau and Wong, 1996)
- UCS= (7-10) x Is (Melborne mudstone) (Agustawijaya, 2007)

ISRM (1985) warns of possible error due to significant scatter in the data. This scatter is also reflected in correlations published by, amongst others, Forster (1983), Bowden, et al. (1998) and Agustawijaya (2007). Additionally, Kanji (2014) recognised that soft rock samples generally deform significantly before failure, which may affect the measurement. However, due to the cheap and simple nature of this test, it is commonly used in industry (Johnston, 1991).

To evaluate the use of these correlations, some of the trends provided above have been plotted with actual data of sandstone, mudstone and weathered granite. These points represent results from a point load test taken on core within 0.5 m of the corresponding UCS test. Additionally, point loads below 1 MPa on rock specimens with a UCS value greater than 20 MPa were excluded as they are assumed to represent failure on existing discontinuity planes. This plot is shown in Figure 8.

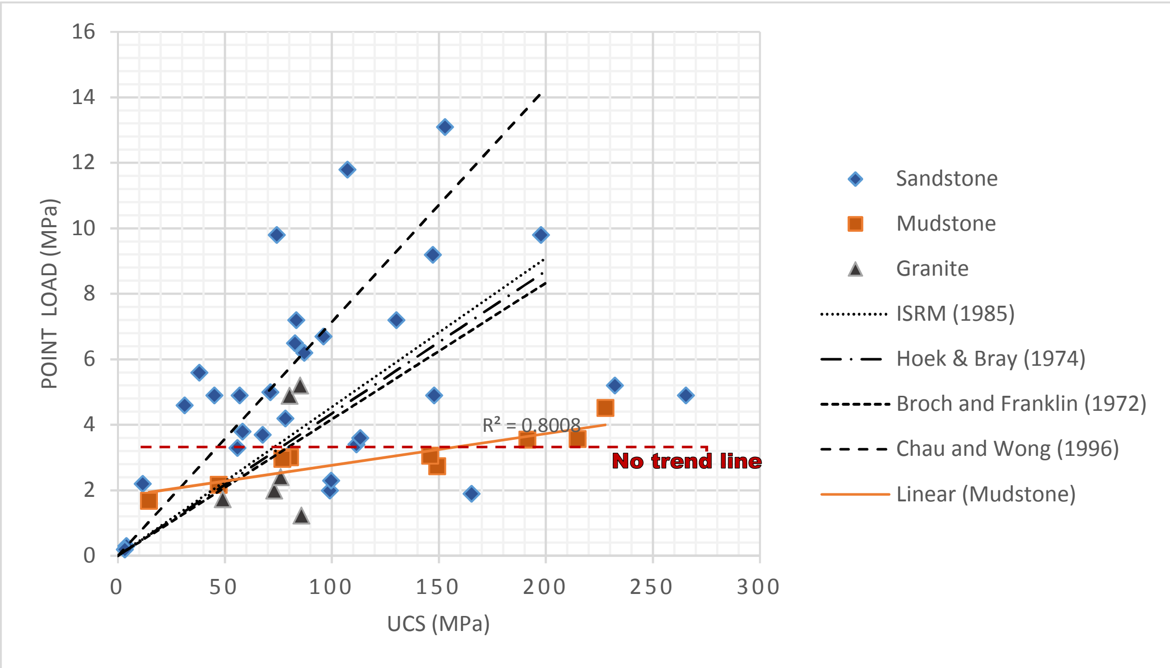


Figure 8: Correlation between Point Load and UCS

The results suggest that, contrary to general belief, the correlation is mostly affected by the rock type and not variation in rock hardness. Here, the weathered granite follows the correlation suggested by the ISRM (1985), Hoek and Bray (1974), as well as Broch and Franklin (1972) respectively, all of whom considered hard rock samples. An improved estimate for sandstone (with UCS values below 200 MPa) is obtained by the correlation by Chau and Wong (1996), which is initially proposed for soft rock. However, for estimates on mudstone, the point load test is not recommended.

Although there appears to be a relationship between point load and UCS measurements, similar point load measurements are associated with a wide range of UCS values (70 – 200MPa). In order to verify the effect of anisotropy on the point load - UCS strength relationship in mudstone, diametrical test results were compared to axial test results. Both showed the same correlation shown in Figure 9. Thus, it appears that anisotropy is not responsible for poor prediction of the UCS strength from point load test results.

Additionally, a weak correlation between UCS and point load estimates for mudstone was observed within several other published results, one of which is illustrated in Figure 9.

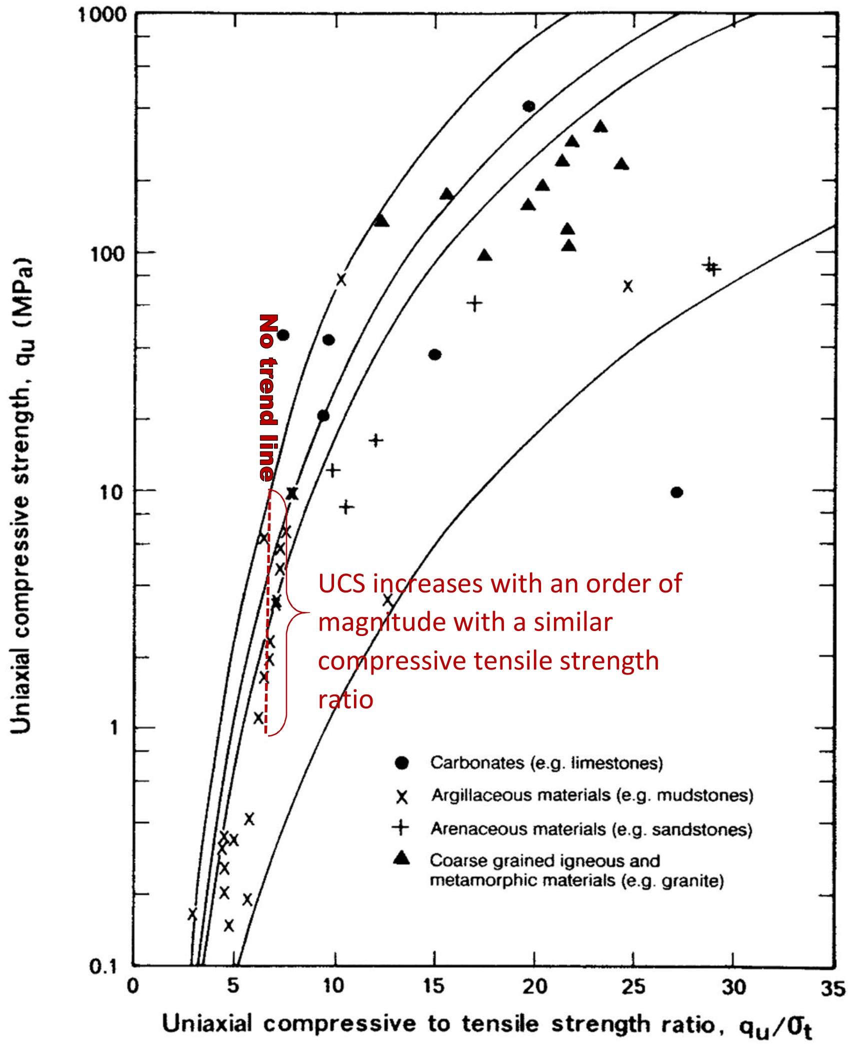


Figure 9: Alternative correlation between Point Load and UCS (Johnston, 1991)

Punch Tests

Stacey (1980) presented a simple and inexpensive test whereby a thin rock plate is tested in double shear by applying a compressional force. The shear strength is calculated by dividing the applied force at failure by the area of the sample through which shearing took place. By using disks cut at different angles, the anisotropy within the intact rock material may also be evaluated. A result from Stacey's study is included in Figure 10.

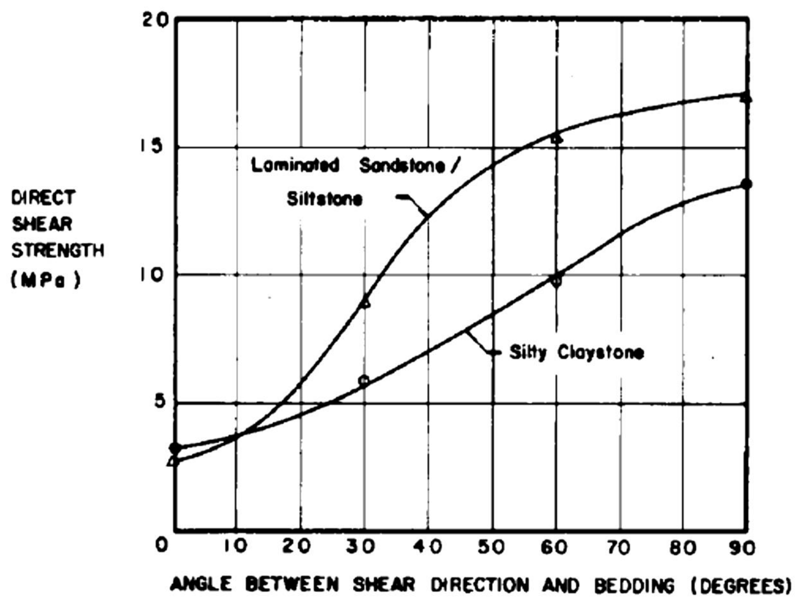


Figure 10: Punch test results on laminated rock (Stacey, 1980)

These results indicate that the strength is predominantly influenced by the bedding / joint planes where the sample is cut at an angle within 15° of the discontinuities. The maximum strength is measured where the disk is cut at a 90° angle to the bedding plane.

ISRM (2007) proposed a standardised test method for the punch test apparatus used for an indirect determination of the UCS value. The recommended correlation between punch test results with UCS and tensile strength is shown below:

$$UCS = 5.1BPI_c$$ **Equation 6**

$$\sigma_t = 0.68BPI_c$$ **Equation 7**

Where

- BPI_C is the Punch test result; and
- σ_t is the Tensile strength.

Gokceoglu and Aksoy (2000) consider a correlation of 5.25 for clay bearing and stratified rock types. Their finding is given in Figure 11.

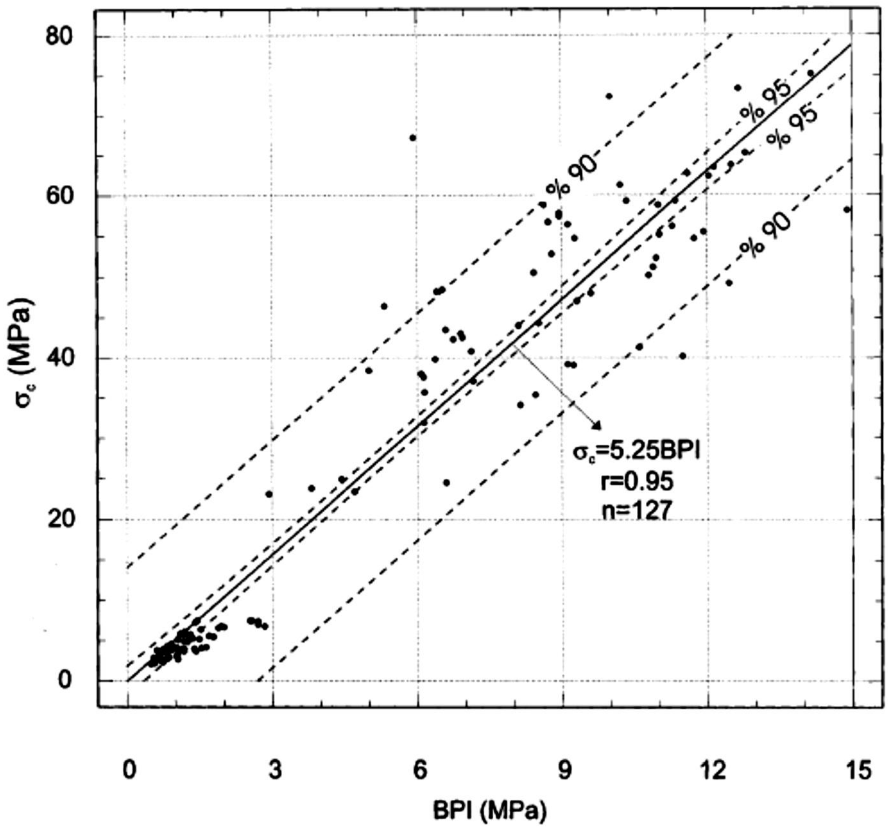


Figure 11: Punch test results on clay bearing and stratified rock types (Gokceoglu and Aksoy, 2000)

The standardised loading rate for both the point load and punch test is such that failure occurs within 10 – 60 seconds. The effect of the loading rate for these alternative testing methods could not be found within the available literature.

Additionally, although the punch test has the potential to provide meaningful estimates for design, it is not commercially available in South Africa.

<u>Vane shear</u>

The vane shear test was originally developed to test the undrained shear strength of soft clay. The measurement is dependent on the dimensions of the equipment and rate of shear (Das, 2002). This technique is used extensively in the UK, where the area is covered by significant clay deposits.

Wiid (1981) presented a modified vane shear test which extended this technique, using a robust design, to test soft rock up to UCS values of 20 – 30 MPa. The equivalent shear strength is calculated by assuming shear over a circular area:

$$\tau = 2T(\pi d^2 D)^{-1} \qquad \textbf{Equation 8}$$

Where

- T is the maximum torque;
- d is the diameter of vane apparatus; and
- D is the depth of embedment.

Although the study revealed promising results, this technique has not been adopted in commercial practice. However, due to the potential of this technique, it is considered as the focus for the experimental study presented in Chapter 3 of this report.

Schmidt Hammer

The Schmidt Hammer is a portable device that measures the rebound hardness of the rock (Ulusay 2015). There are essentially two different types of Schmidt hammers available including (Mol, 2016):

- N-Type; high impact hammer
- L-Type; low impact energy release device that is suitable for surfaces less than 100mm thick.

Measurements with the Schmidt Hammer are, however, affected by neighbouring fractures within the rock outcrop (Mol, 2016). Various correlations between the Schmidt Hammer and UCS have been proposed, one of these correlations (Hoek, 2007) is included as Appendix A in this report.

Absorption

Many researchers, including Onodera and Asoka Kumara (1980), Ulusay et al. (1994) and Tiryaki (2006) showed that the geotechnical characteristics of rock, including rock strength, are a function of the rock texture and mineralogy. This realisation sparked research on possible reasonable strength estimates based on physical rock properties.

Absorption is a simple, inexpensive test that can be correlated to porosity. Porosity has been found to be an important index parameter for soft rock, with the general trend comprising an increase of strength with decreasing porosity (Kanji, 2014). This trend is also found in a study by Agustawijaya (2007), considering the effect of weathering on rock strength as seen in Figure 12.

Additionally, researchers such as Bell (1978) showed that the decrease in UCS due to water content is proportional to the porosity of the rock.

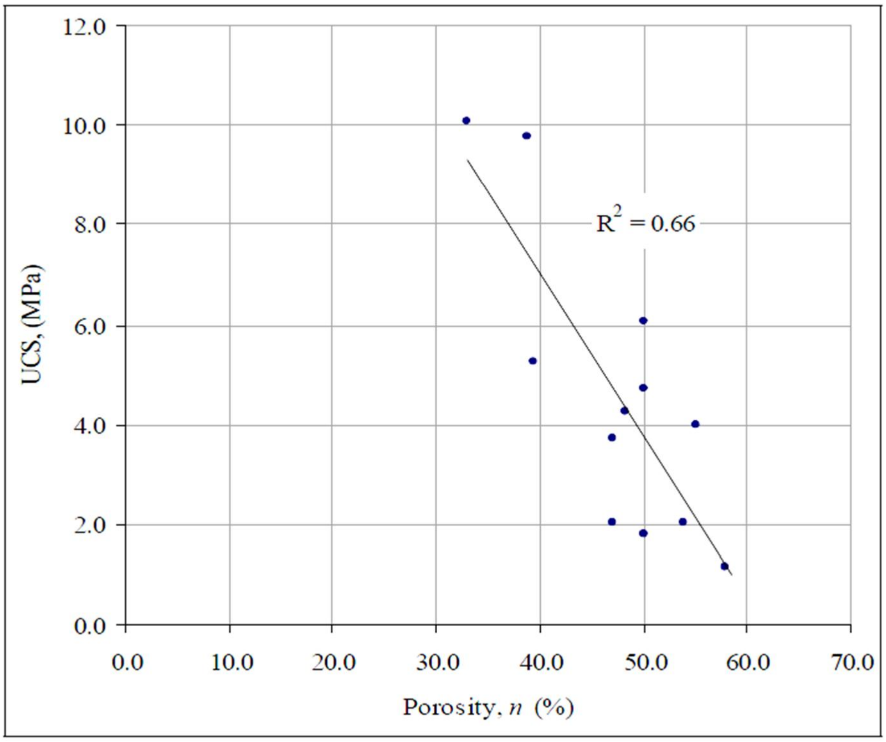


Figure 12: Correlation Between UCS and Porosity (Agustawijaya, 2007)

<u>Fracture toughness</u>

Within the field of fracture mechanics where brittle behaviour is explored, an additional inherent material property has been identified. This is commonly referred to as 'fracture toughness' and as the name suggests, it describes the material's resistance to crack propagation. Although this parameter is not traditionally used for strength estimation, it may be the limiting factor for the strength of a rock before failure and is, thus, considered within this study.

The material resistance to crack propagation is described as the energy required for a fracture to propagate, which is proportional to the area below the stress strain curve (Nara et al., 2012).

Fracture mechanics considers an energy concentration at sharp cracks (or defects within the rock material), causing the crack to grow. If the fracture toughness is less than the yield strength, the cracks will grow when the applied load results in an energy concentration beyond the inherent rock toughness. This result in brittle failure at a lower strength. Should the fracture toughness exceed the yield strength, plastic deformation will result, with a distribution of energy throughout the material and the fracture / defects will have little effect on the rock strength (Tiryaki, 2006).

Work by Bieniawski (1967), estimated that crack initiation and stable fracture propagation occurs from approximately 35% of the UCS strength, with critical energy release from approximately 80%.

Atkinson et al. (1986) proposed a toughness index, which is an indirect measure of the rock fracture toughness based on a simple energy balance of measurable index parameters and is estimated by:

$$T_i = \frac{\sigma_c^2}{2 \times E}$$ **Equation 9**

Where

- T_i is the toughness index,
- σ_c is the UCS, and
- E is the Young's Modulus

A toughness index of 27 is considered the limit of the rock to be cuttable (Tiryaki, 2006).

Considering the excessive energy required to cut a material with a toughness index exceeding 27, it is reasonable to assume that this parameter distinguishes between a rock mass where failure through the material is possible (considered as a soft rock mass) and where failure through the material is unlikely (hard rock mass).

To assess the upper limit (ie. hardest intact material) for a soft rock mass, the toughness index value, for 163 laboratory samples were determined and plotted against their corresponding UCS value. These results, shown in Figure 13, indicate that the limit is affected by the rock composition, with general limits:

- Mudstone with a UCS up to 100 MPa is considered soft; and
- Sandstone and crystalline rock is soft up to 150 MPa.

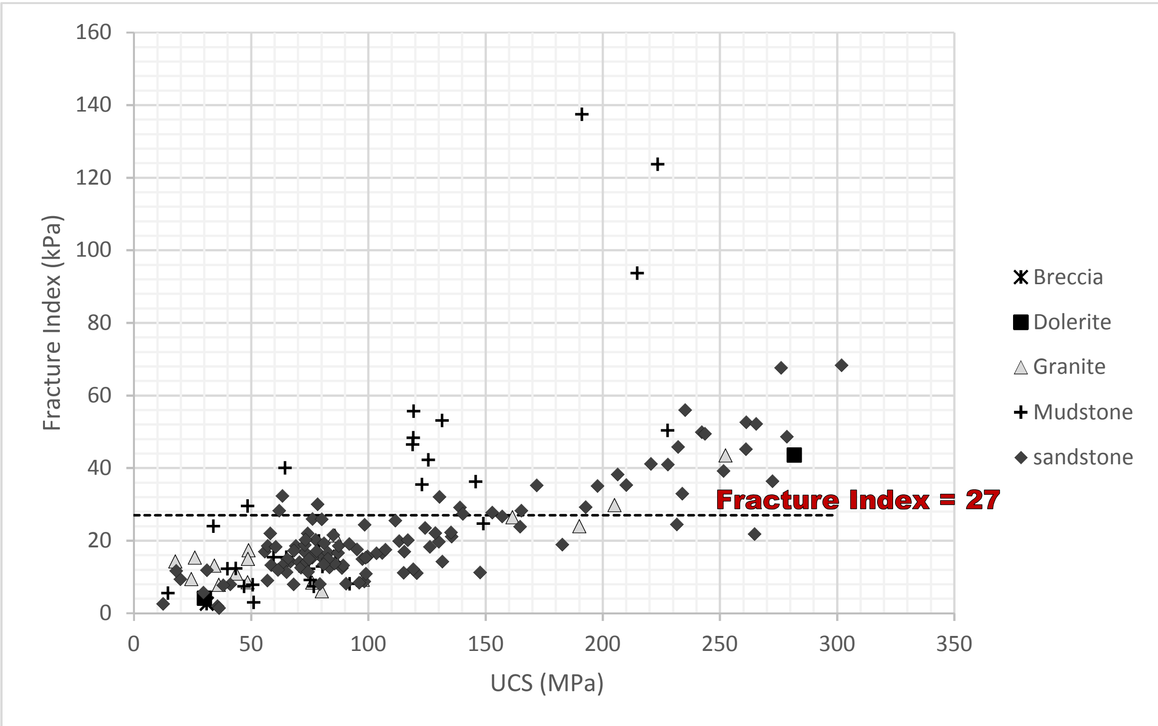


Figure 13: Correlation between Point Load and UCS

Experimental results from Nara et al. (2012) indicate that the fracture toughness decreases with an increase in the degree of saturation of the rock sample. Since the elastic range also decreases, the general limits above should, however, not be significantly impacted by the degree of saturation.

2.1.2 Soft rock related to material with high clay content or other unstable minerals

The materials in this group disintegrate or slake (rock breaks up into silt to clay type deposit) within the design life of an engineered structure, when exposed to water or climate change (Nickmann et al., 2006). Affected deposits comprise of mainly sedimentary rock (e.g. sandstone, mudstone or shale) and to a lesser degree basic crystalline material.

Experimental studies generally show that mudstones with less than 56% quartz are vulnerable to slaking. Additionally, the durability of vulnerable deposits is mainly associated with the clay type and proportion within the rock (Lashkaripour and Boomeri, 2002).

Currently, rock durability for potential construction materials is assessed mainly by the wetting and drying test or the more aggressive slake durability test.

The standard slake durability test (ASTM D 4644-87, 2001) comprises of oven dried samples placed in a 2.0 mm square-mesh cylinder, that is then submerged in distilled water and rotated at a standard speed for 10 minutes. The sample is then dried in the oven and the material retained within the cylinder measured. This is one cycle.

Santi (2006) defined this group of soft rock as material with a slake durability <90% (done with standard 2 cycles)

However, a study by Bell et al. (1997) indicated that 2 cycles is generally insufficient for very fine-grained material such as shale. Four cycles provide a better indication.

Although these estimations indicate that rock is potentially prone to disintegration / slaking, it does not distinguish between spontaneous decay and slow degradation over months. Nickmann et al. (2006) thus proposed a classification system based on a wetting and drying test.

This test comprises of gravel size fragments between 25 mm – 50 mm that are covered by water, left to soak for a period of time and then dried in the oven. This is one cycle.

Based on the test results, the disintegration tempo is defined as follows:

- *Spontaneous decay* – Less than 25% of the mass remaining after the 1st cycle;
- *Rapid decay (days*) – 25 to 90% of the mass remaining after the 1st cycle;

- *Moderate decay (weeks)* – More than 90% of the mass remaining after the 1st cycle, but less than 50% of the mass remaining after the 3rd cycle;
- *Slow decay (months to years)* – More than 90% of the mass remaining after the 1st cycle, with 50 - 95% of the mass remaining after the 3rd cycle.

Various attempts have also been published with correlations between the physical rock properties and durability. One of the most popular being rock strength (using point load and Schmidt hammer test results).

Although some correlation between rock strength and durability was noted (e.g. soft rock tends to be less durable), this does not provide an exclusive indication. In South Africa, mudstone with UCS values more than 100MPa can crumble into gravel size fragments when exposed to the atmosphere for a couple of months.

Rock durability is significantly affected by its mineral composition. Unstable minerals include iron sulphides with chloride and clay minerals, especially when sodium is present (Walkinshaw and Santi, 1996).

The sulphides (ie. pyrite) react with the groundwater, increasing its acidity, which in turn dissolves the chlorides. The resultant progressive weathering is illustrated in Figure 14, where the shale samples contain the undesirable minerals. However, it should be noted that, although the rock weathering and strength are affected, both rock specimens would have tested as durable aggregate.

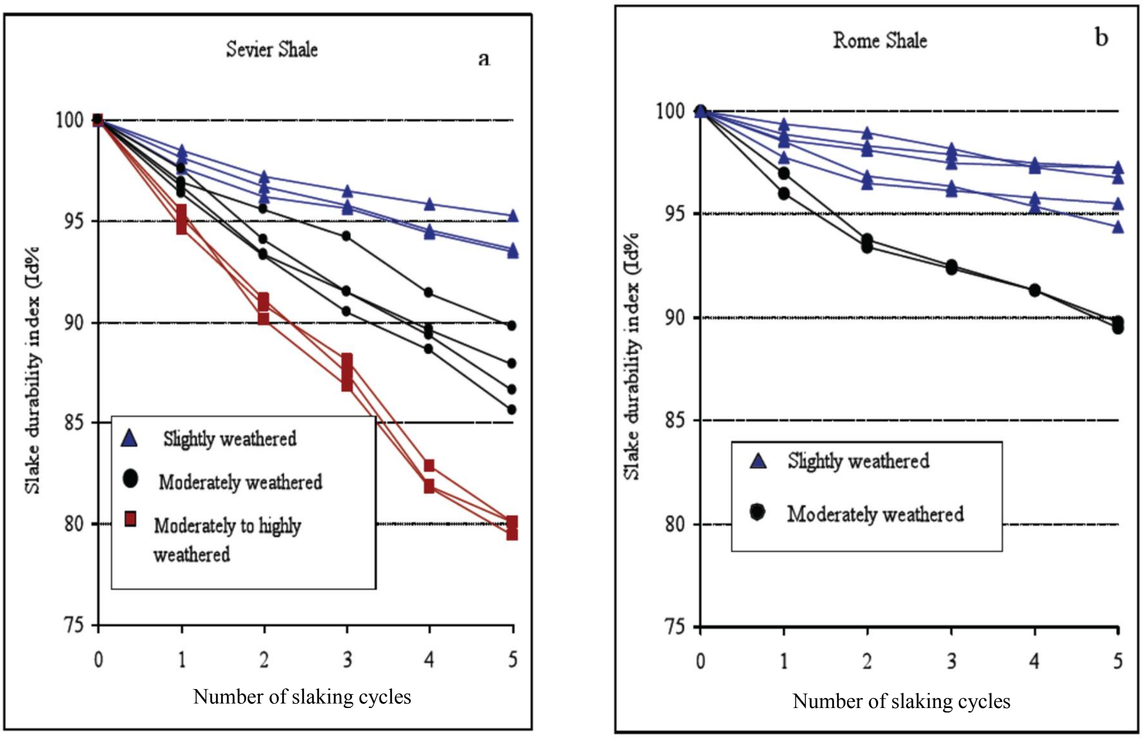


Figure 14: Durability of shale containing iron sulphides (Nandi et al., 2009)

Additionally, Walkinshaw and Santi (1996) found that some shale has dispersive tendencies and reacts with alkaline water. However, no analysis on rock with dispersive tendencies were provided. It is nevertheless reasonable to assume that rock prone to disintegration due to dispersion comprise of dispersive material.

In South Africa, dispersive soil is mostly related to sedimentary rock deposits of the Beaufort Group and Stormberg Group within semi-arid climatic conditions (Weinert's N value between 2 and 10) (Elges, 1985). It is interesting to note that soil formations in this region mostly undergo physical disintegration without significant chemical alteration.

Parker (1963) noted that dispersive soil is associated with one or more of the swelling clays present in its composition. Levy and van der Watt (1988), also recognised the direct correlation between the expansiveness of the type of clay, and the dispersive nature of the soil (i.e. soil with smectite is more dispersive than soil with illite). Additionally, sodium (Na^+) is a highly dispersive agent and if this is present in the clay, the soil is likely to be dispersive (Bell and Walker, 2000).

The strength of the rock decreases due to slaking. Experimental results, showing the reduction in strength due to slaking, is presented in Figure 15.

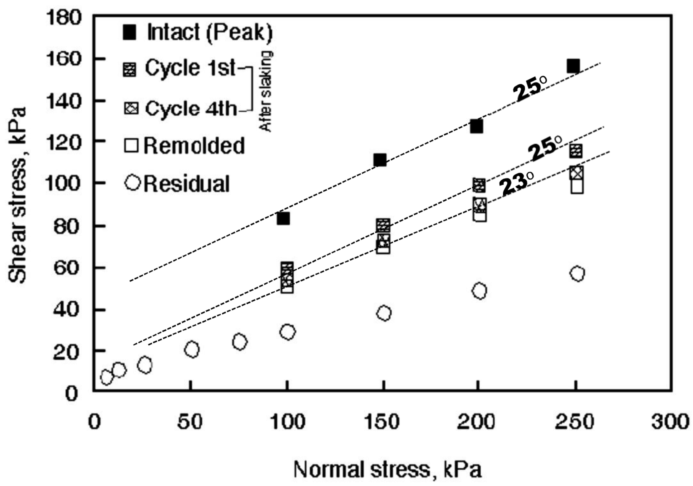


Figure 15: Shear strength estimation at various degrees of slaking (Bhattarai et al., 2006)

The results show material that is prone to spontaneous decay. Although there is a slight reduction of the internal angle of friction, it is the cohesion that is mainly affected.

Based on these results, the long-term strength estimate would have a similar angle of internal friction, with insignificant cohesion. The reduced strength is also related to the exposed zone and not the entire rock (mass).

The Schmidt hammer is one of the techniques that measure the rock strength along weathered surfaces. Due to weathering, the strength at the surface is less than the in-situ rock material. When fresh surfaces are tested, the measurement should indicate a similar result than UCS tests in the same material. As the material becomes more weathered, the difference between the measurements increases. Singh and Gahrooee (1989) proposed a classification for the degree of weathering and this is represented in Figure 16.

Description	Weathering coefficient
Fresh–slightly weathered	$(\sigma_c^a/JCS^b)<1.2$
Moderately weathered	$1.2<(\sigma_c/JCS)<2$
Weathered	$(\sigma_c/JCS)>2$

[a] σ_c: uniaxial compressive strength of unweathered rock.
[b] JCS: joint strength determined by the Schmidt hardness test.

Figure 16: Weathering Classes (Singh and Gahrooee, 1989)

2.1.3 *Soft rock due to the presence of weathering*

Weathered rock, sometimes referred to as 'saprolite', describes a material where the rock structure is still discernible in an almost soil like material and where many of the less durable minerals have been altered to clay (Nandi, et al., 2009).

Where the weathering is uniform, the behaviour of the material is like soil, but sometimes the original rock structure or preferential weathering along bands of less stable minerals result in zones of weakness. This material resembles a rock mass (Walkinshaw and Santi, 1996). Generally, weathering results in the reduction of strength and an increase in failure strain. These rocks are also more sensitive to changes in moisture content (Wong et al, 2016).

Highly weathered rock material falls in the lower limit of soft rock that in itself has significant engineering difficulties. One of the main challenges is the collection of representative samples for testing and the availability of suitable testing techniques.

An example of this is weathered granite, where the SPT spoon refuses and the Shelby tube is unable to penetrate the material. On the other hand, recovery, due to disturbance during drilling resembles a gravelly sand. When the practitioner is fortunate enough to obtain a reasonable representative sample, this is often too soft to test using rock mechanics techniques, but too hard to cut and test in equipment designed for soil.

However, various researchers have found that the internal friction angle for a remoulded sample resembles the friction angle of the in-situ rock. The cohesion is lower due to the breaking of in-situ bonds.

This decrease in strength is more significant in sandstone or crystalline material compared to mudstone, as illustrated in Figure 17.

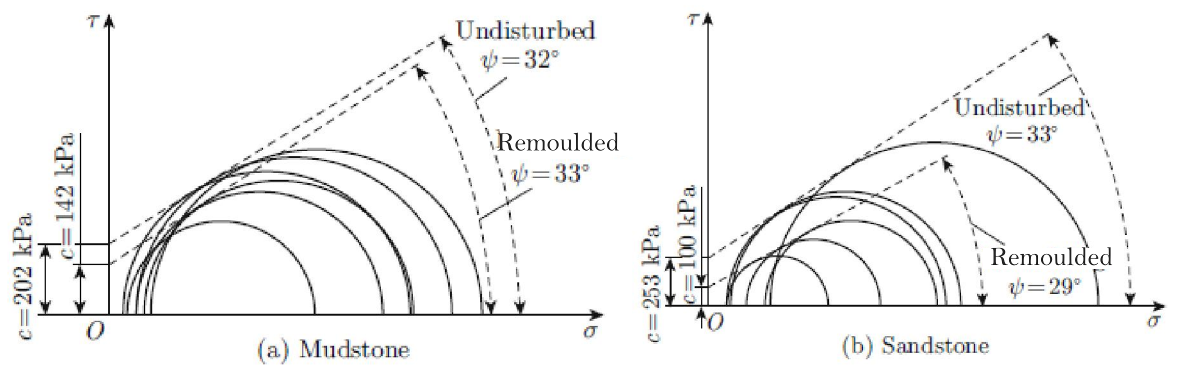


Figure 17: Comparison between undisturbed and remoulded rock (Wang, et. al., 2013)

2.1.4 *Soft rock due to excessive deformation*

A rock (mass) may be strong enough not to fail under a given stress, but the strain may still affect the serviceability of the development. Thus, the behaviour of the material up to failure can be as important as the strength itself.

In this study, the resistance of a material to deform due to stress (which represents the relationship between an applied stress and resultant deformation) is referred to as rock stiffness.

Various studies identified the following general trends (Clayton, 2011):

- Stiffness is anisotropic;
- Stiffness decreases with increasing strain; and
- Stiffness increases significantly with depth.

The increase in rock stiffness with depth is related to the decrease in the elastic properties of the rock as the pressure and temperature increases (He, 2014). Chinese scholars (He, 2014) deal with this variation by considering a critical pressure and temperature. They distinguish between a geological soft rock (referring to all the characteristics described above) and an engineering soft rock, explaining the conditions where even a geologically hard rock behave as soft material.

Hoek (1999) found that geologically hard rock behaves as soft material when the UCS is less than approximately one third of the in-situ stress acting on the rock (Scott, 2007).

There are fundamentally two types of stiffness values that can be recorded for a given rock (mass) namely, dynamic stiffness (at very small strain measured when a wave travels through the material) and static stiffness (the rock stiffness at various strain levels), estimated in the laboratory (UCS with strain gauges, or triaxial testing) or in the field (plate load, flat jacks, etc.). The correlation between dynamic and static stiffness varies between 1:1 and 1_{static}:$2_{dynamic}$ as illustrated in Figure 18.

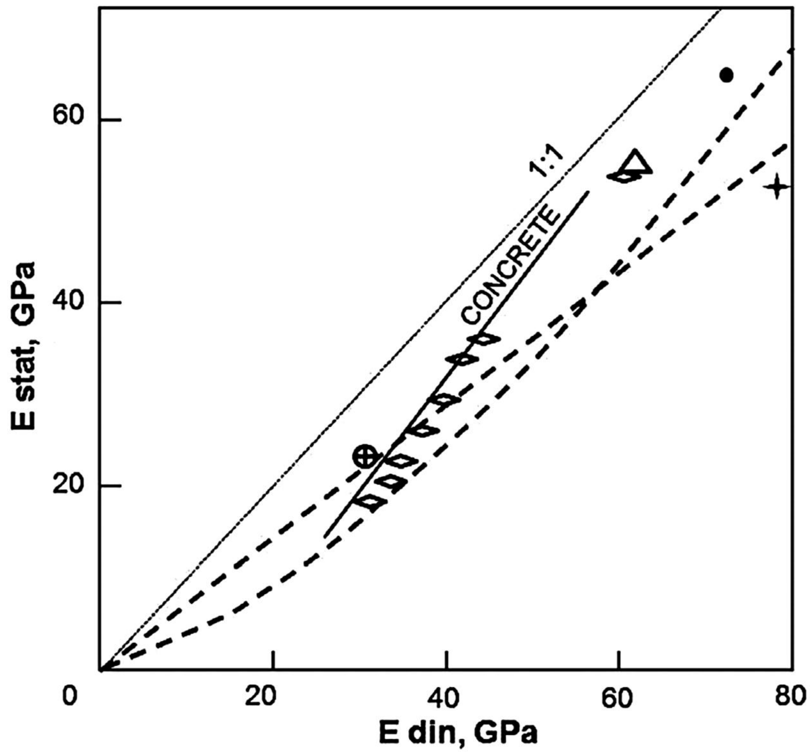


Figure 18: Relationship between static (E_{50}) and dynamic moduli for intact rocks (Kanji, 2014)

Additionally, one could determine dynamic and static values for either shear (represented by the shear modulus "G") or volumetric stiffness (represented by Young's modulus "E").

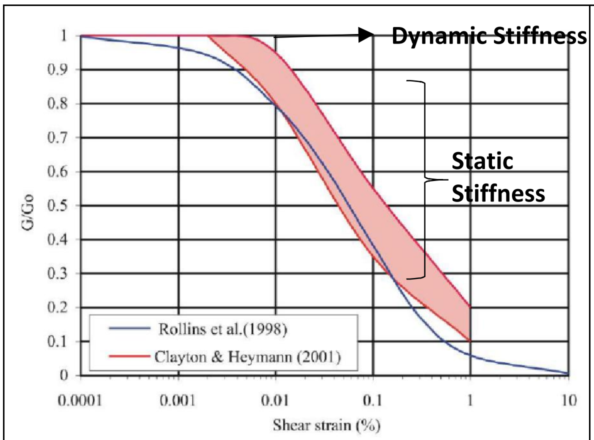
Considering a linear elastic medium, there is a direct relationship between shear and volumetric stiffness, which is represented by:

$$G_{50} = \frac{E_{50}}{2\,(1 + v)} \qquad \textbf{Equation 10}$$

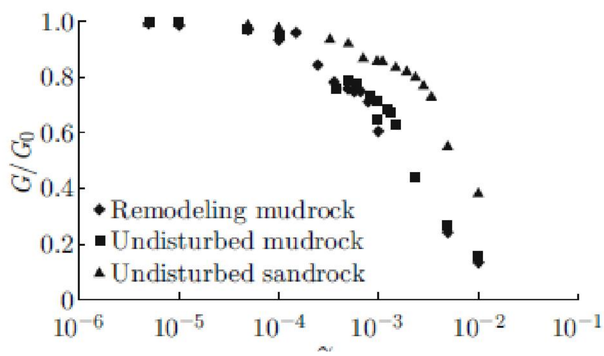
Where

- G_{50} is the Shear stiffness at 50% strength;
- E_{50} is the Volumetric stiffness at 50% strength; and
- v is the Poisson's Ratio.

For soil and rock material, the dynamic shear stiffness gives an estimate of the maximum stiffness of the material (with linear stiffness estimate up to a maximum axial strain of 0.004%), whereas the static shear stiffness represents a lower value as the material may have already yielded. This is shown in the following Figure:



A: Experimental test results for clay and chalk samples (Clayton and Heymann, 2001)



B: Experimental test results for remoulded material and undisturbed rock samples (Wang et. al., 2013)

Figure 19: Stiffness of soil and soft rock at increasing strain levels

Additionally, Mair et. al. (1993) realised that the strain level, and therefore the associated shear stiffness softening, are less for retaining walls compared to foundations. Thus, the distribution of stresses influences the magnitude of shear strain for the soil or rock material. Mair et. al.'s (1993) findings is given in Figure 20.

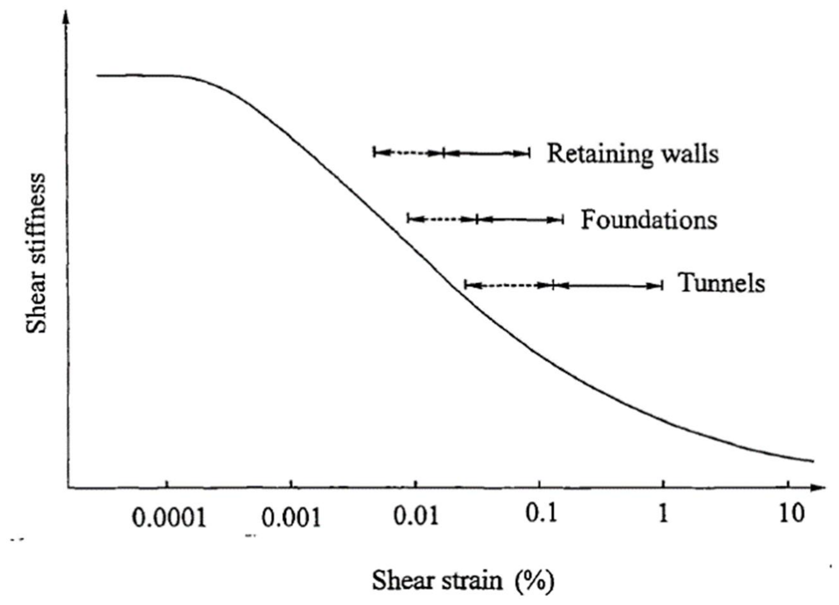


Figure 20: Idealised non-linear stress strain behaviour (Mair et. al., 1993)

Hardin and Black (1968) showed that the distribution of stress does not have the same impact on dynamic stiffness measurements as this is strongly affected by mean effective stress, but remains virtually independent of deviatoric stress.

Many researchers, including Deere and Miller (1966), as well as Kanji and Galván (1998), identified a correlation between rock strength and stiffness. Available data on sandstone and mudstone have been plotted over the identified relationship and is shown in Figure 21.

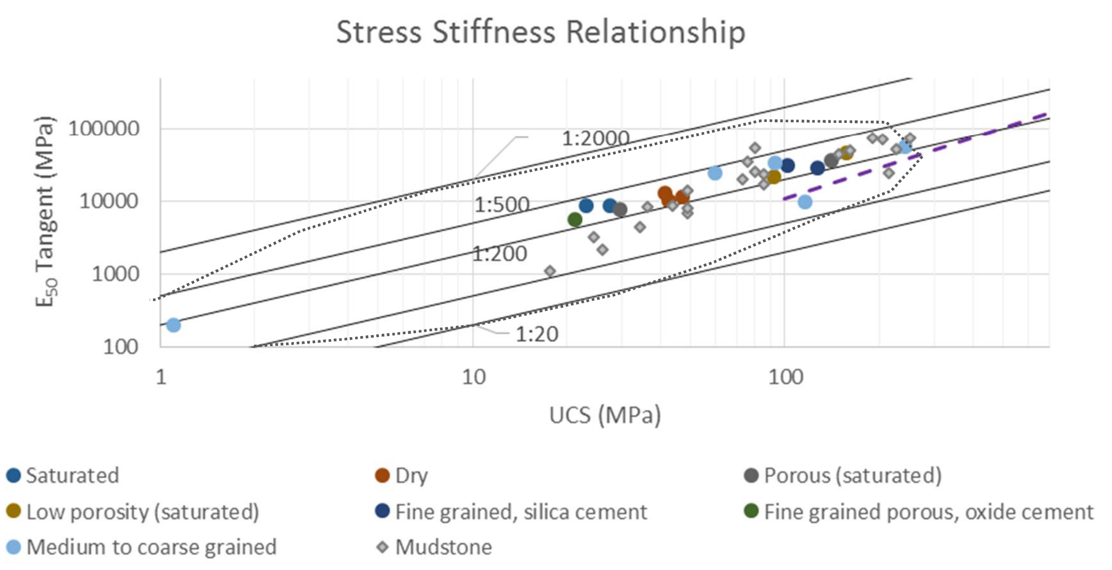


Figure 21: Stress – stiffness relationship (Kanji, 2014)

The results above confirm a direct relationship between rock strength and stiffness that seems to be independent of the physical rock properties, including rock type, grain size, porosity, cementation (bonding agent) and the degree of saturation.

According to a graph, published by Hoek and Diederichs (2006), indicating the strength-stiffness correlation for various rock types, the mudstone, like the sandstone, has an average to low modulus ratio. This is shown in Figure 22.

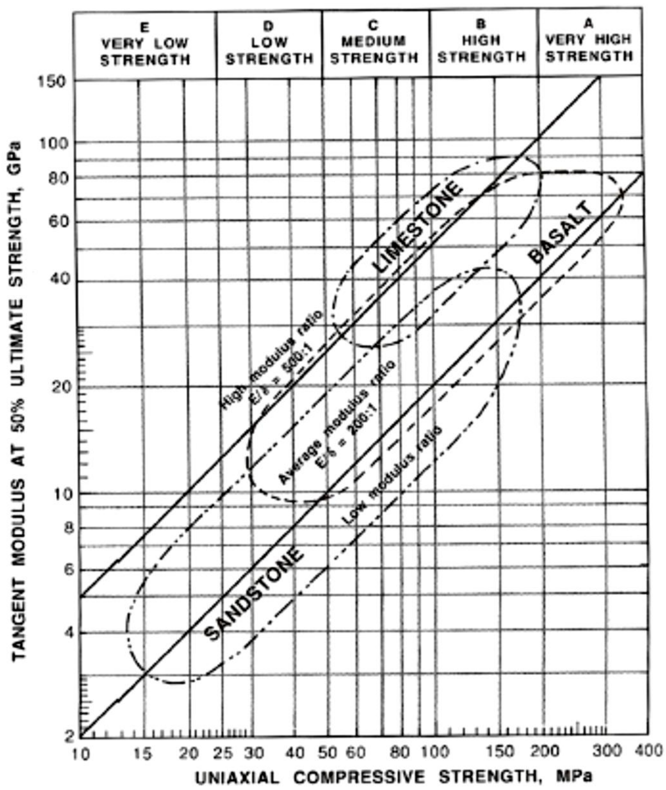


Figure 22: Rock Modulus Values (Hoek and Diederichs, 2006)

The most common and inexpensive way to measure static stiffness is through the Uniaxial Compression Modulus (UCM) Test in the laboratory.

<u>UCM Tests</u>

This index test comprises of a standard UCS test, with strain gauges measuring core deformation during testing. Thus, it provides an additional static stiffness estimate. The E Modulus represents the axial deformation with increasing stress (strain). As an international standard, the static stiffness is determined during laboratory testing at

50% UCS. Alternative static stiffness values have also been recorded, such as the secant, tangent and average stiffness values. These values may differ significantly as the rock stress-strain behaviour may be non-linear.

In Section 2.1.1, the effect of the loading rate on the measured strength was considered. Rock stiffness is also affected by loading rate, considering that the undrained modulus is around 15% higher than drained modulus (Galera et al., 2005).

Dynamic stiffness is mostly estimated through geophysical surveys.

Geophysical surveys

Santi (2006) describes a soft rock as a material in which a seismic P-Wave velocity of less than 2100m/s is measured. This indicates that seismic waves tend to travel slower in softer ground conditions.

These techniques either measure body waves (through a combination of resonant column or bender elements in the lab and downhole or cross hole surveys in the field) or surface waves (Continuous Surface Wave (CSW) Tests in the field).

The dynamic shear stiffness (G_0) is generally estimated through the empirical relationship:

$$G_0 = \rho V_s^2 \qquad \textbf{Equation 11}$$

Where

- ρ is the rock density; and
- V_s is the Shear (S) wave velocity.

During an investigation for a potential windfarm in the Northern Cape, the stiffness of the Karoo Mudstone was estimated with a combination of CSW tests and measurements on core samples in the laboratory. For this study, the shear (G) modulus at 0.1% strain (estimated from CSW test results with Equation 11 and Figure 19) was converted to an equivalent Youngs (E) modulus. These are presented as upper and lower limit profiles up to a depth of 10m below surface in Figure 23.

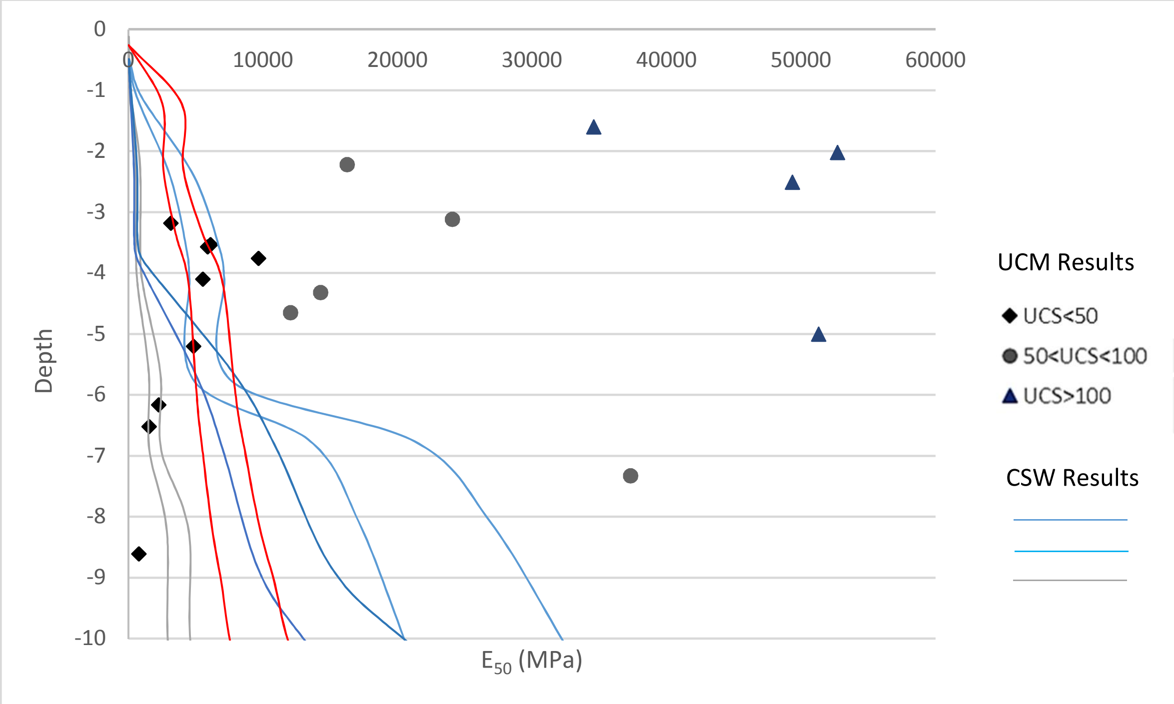


Figure 23: Representation of stiffness test results

The results show that the estimates from the CSW test results correspond with the soft rock. CSW profiles represent an average estimate that is influenced by the discontinuity network, which masks the higher stiffness in hard rock zones.

Like the Point Load test, several researchers (including Christaras et. al. (1997), Yasar and Erdogan (2004), Entwhistle et. al. (2005), Tercan et al. (2005), Sharma and Singh (2008) and Altindag, 2012) have published empirical correlations between P-Wave velocity and UCS values. It was found that these correlations are mainly influenced by the rock type. Two researchers, in particular, considered a general trend for a range of material that include both sandstone and mudstone and hence selected for this study.

It should be noted that empirical relationships are based on real data from a specific material under a specific set of boundary conditions. It is thus not the intention of this study to augment the selected correlations, but it is merely seen as relevant to investigate the general trends in soft rock. In practice, it would be beneficial to use a correlation where the material and boundary conditions resemble the site. Calibration, where possible is also highly recommended.

General trends include:

- UCS = 7.1912 Vp + 26.258 (Tercan et al., 2005)
- UCS = 12.746 $Vp^{1.194}$ (Altindag, 2012)

UCS estimations based on the CSW test results and selected trends were plotted with UCS values from laboratory test results. Additionally, 30% of the measured strength from the lab estimates were subtracted to account for scale. This is shown in Figure 24.

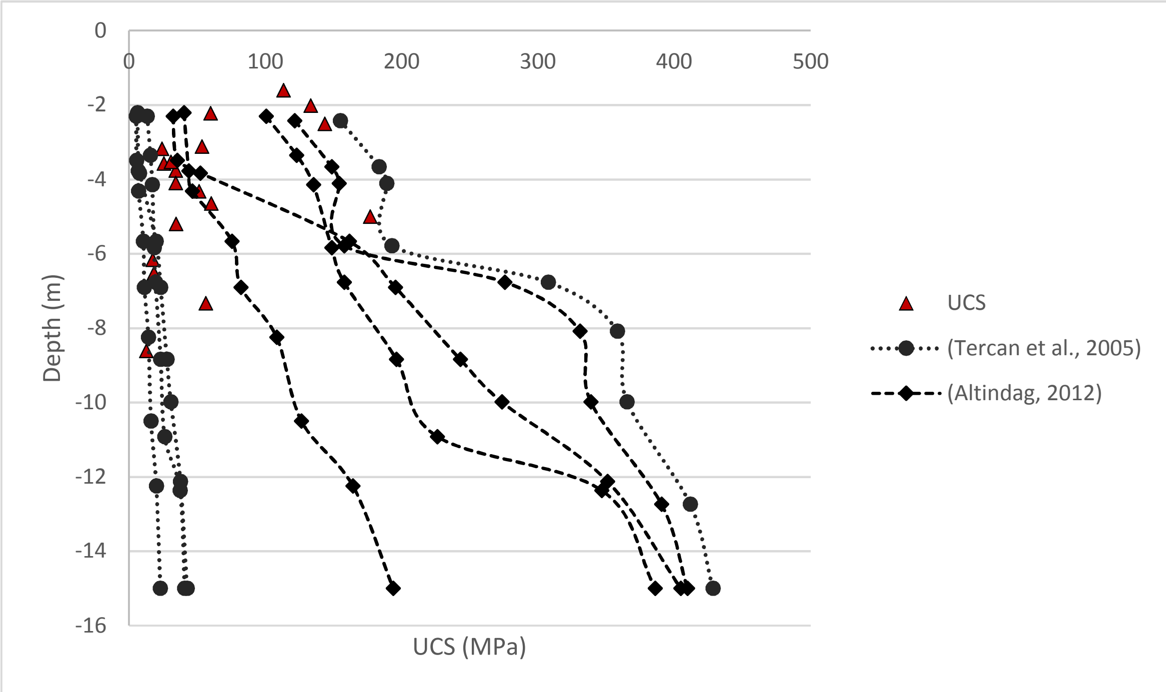


Figure 24: UCS Estimations based on CSW and laboratory test results

The results show that the exponential function proposed by Altindag (2012) is more representative of the UCS measurements in fractured mudstone as it shows a direct correlation with the rock stiffness presented in Figure 23.

The results in Figure 24 also show a suitable correlation between the reduced UCS estimates from the laboratory test results and estimates from the CSW Test results under low confining pressures.

It should be remembered that UCS estimates based on CSW test results are vulnerable to scatter due to the effect of water on the P-wave velocity. The water content and general groundwater conditions of the material tested is often not known.

Effect of time

Rock creep refers to deformation of the rock material under constant stress over time. It is generally accepted that creep is unlikely if the load/stress level does not exceed a certain threshold value. This is sometimes referred to as the long-term strength of rocks (Bieniawski 1970).

Available testing techniques include Uniaxial, Triaxial Compression or Brazilian Indirect Tensile Creep Tests. This is time consuming and thus not commercially practical. Thus, industry rather relies on models based on available research.

Creep is generally expected where the stress level is about 40 – 60% of the rock strength (Nyungu, 2013). Additionally, according to Ryder and Jager (2002), the creep rate in rock is dependent on the magnitude of the deviatoric stress (σ_1 - σ_3) and not the individual magnitudes of σ_1 and σ_3 (Nyungu, 2013).

2.2 Fractured rock mass

A rock mass generally comprises intact rock material that has been divided into fragments by a network of discontinuities (such as faults, joints, bedding planes etc.). The behaviour of the rock mass is thus controlled by a collaboration between the characteristics of the rock material and fracture network, which are both affected by the surrounding environment (confining pressure, water, etc.) (Scott, 2007). Compared to intact rock material, a rock mass is characterised by decreased strength with very limited tensile strength (Barton, 2013). The discontinuity network may have an apparent tensile strength due to the mass geometry.

The elastic Young's Modulus (describing the intact rock material resistance to deformation under a given stress) is not strictly applicable to rock masses. Though the rock behaves in a linear manner, it does not return to its original state when it is unloaded and the stiffness decreases with increasing crack density (Kachanov, 1992). Thus, researchers generally refer to an equivalent 'deformation modulus' for rock masses (Schultz, 1996).

The relationship between continuous rock mass strength and stiffness is affected by the mode of failure, with gradients on the log–log plot ranging between 1.4 for sliding and 1.8 for shearing. By considering all possible modes of failures, an average gradient of 1.6 has been found, as shown in the Figure below (Ramamurthy, 2004).

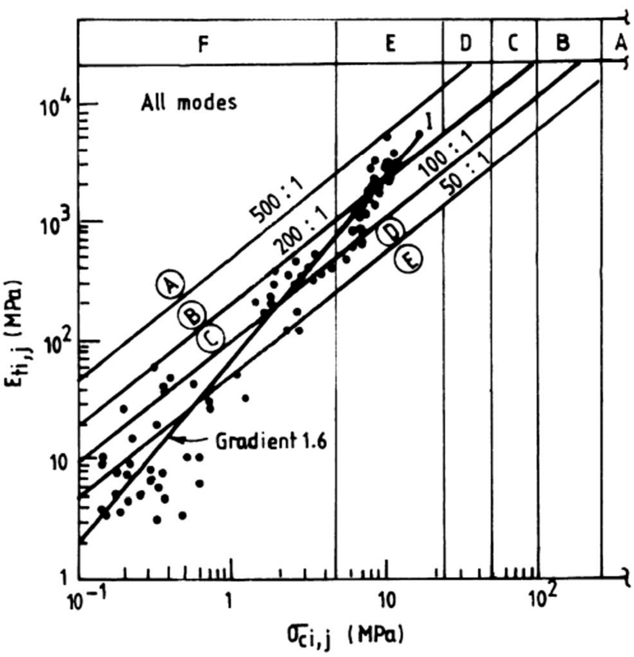


Figure 25: Impact of jointing on rock strength – stiffness relationship (Ramamurthy, 2004)

Intensely fractured rock is generally considered a weak rock mass. Additionally, Goldstein et al. (1966) found a direct correlation between the intact material strength and the percentage reduction of strength due to fracturing. Therefore, greater the intact strength of the rock material, the more sensitive the resulting rock mass strength is to the fracture intensity.

Hard rock is generally considered weak due to fracturing when the Rock Quality Designation (RQD) is less than 25% (Santi, 1997).

2.2.1 Weak rock mass under low confinement

Under low confinement, rock mass behaviour is mainly controlled by its discontinuities. The rock mass may either be discontinuous (where stability analysis considers the possible modes of failure within the discontinuity network through either a kinematic or key block analysis), or continuous (where stability analysis considers isotropic behaviour due to the interaction of rock material and discontinuity network). The rock mass response (structurally controlled or mass failures) depends on the scale as illustrated in Figure 26 below.

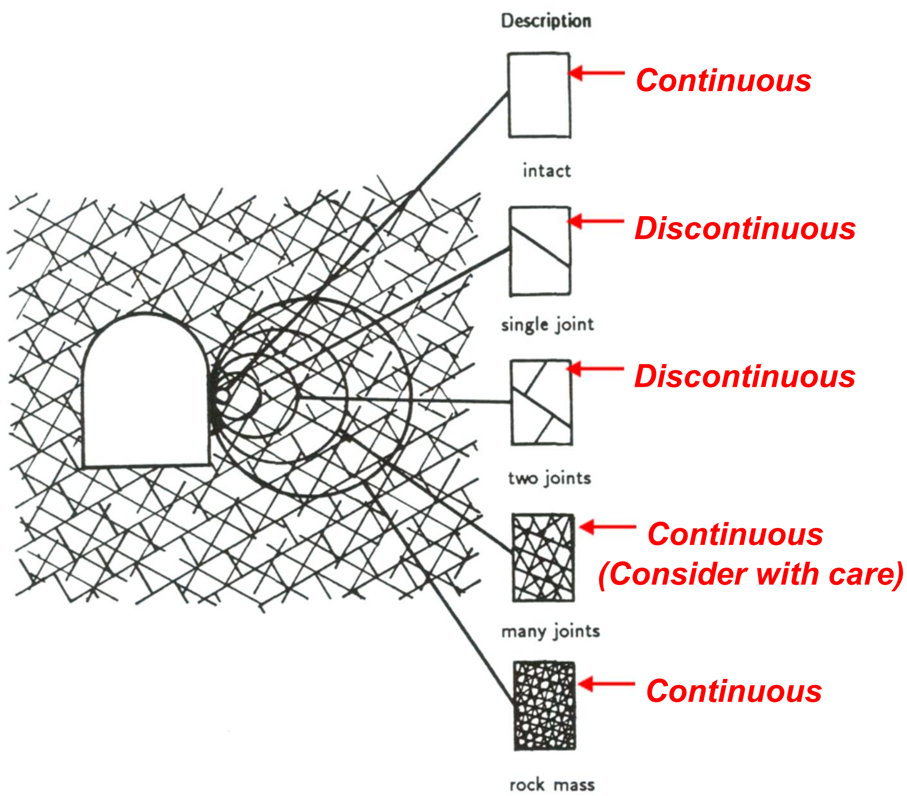


Figure 26: Impact of effect of scale on rock mass behaviour (Hoek, 2007)

As a rule of thumb, a continuous rock mass may be assumed if the structure is 5 – 10 times the discontinuity spacing (Schultz, 1996 and Scott, 2007).

However, it should be remembered that a closely jointed rock mass will still be structurally controlled if one of the discontinuity sets is significantly weaker compared to the other sets within the discontinuity network (Scott, 2007).

In very soft rock masses, the shear strength of the intact rock material generally does not differ significantly from the shear strength of the discontinuities and the effect of scale is considered less important (Russo, 1994).

Additionally, Yang et al (1998) proposed that current continuum models and empirical equations are reasonable for randomly fractured masses, but are limited when highly jointed masses are considered. In these models and equations, the orientation of discontinuities and the resulting non-uniform stresses are not considered.

Influences on the expected behaviour of the rock mass includes:

1. Fracture orientation

   Various researchers found that the angle of discontinuities in relation to the principal stress orientation, in combination with the magnitude of confining pressures, significantly influences the way in which the rock mass will fail (Wasantha et al. 2014). This effect is illustrated in Figure 27.

   Furthermore, under low confinement, a rock mass that is dominated by steep joint sets (at about 70-80°), will most likely fail with additional rotation of the rock fragments (Scott, 2007).

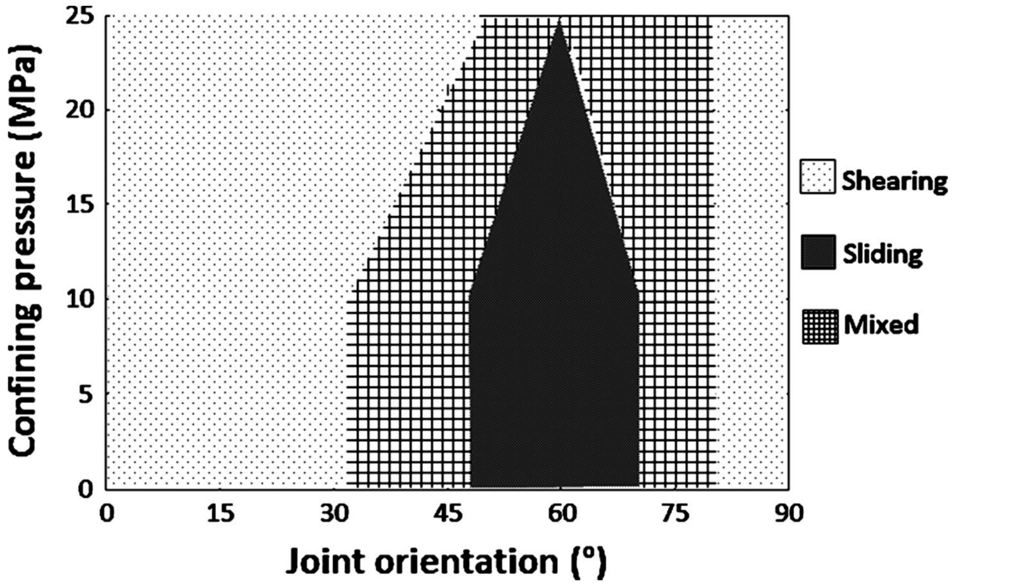


Figure 27: Failure mechanisms under various confining pressures and joint orientations (Wasantha et al. 2014)

A rock mass can either fail along discontinuities (sliding), or through the intact rock (shearing). Sliding occurs at significantly lower stresses compared to shearing. Thus, the strength of the rock mass and the rate at which the rock mass strength increases with an increase in confining pressure is significantly affected by the joint orientation (Scott, 2007).

Some of the studies on the effect of joint orientation on the failure mode consider acoustic emission. Acoustic emission comprises the radiation of elastic waves due to crack formation or plastic deformation in the rock (Li, 2002).

Within one of these studies, acoustic emission monitoring was carried out on samples with varying joint orientations Wasantha et al. (2014). The results showed:

- Vertically jointed masses (0° in relation to σ_1) is similar to those of intact specimens, which is consistent with shearing as the mode of failure (shown in Figure 27).
- Horizontally-jointed specimens and those with joints oriented at 30° show acoustic events at very early stages of deviatoric loading. Damage is thus recorded within the rock that is not related to shearing (Refer to Figure 27). Wasantha et al. (2014) theorise that this may be due to the closing of joints and associated asperity damage.

The effect of joint orientation on the rock mass strength was considered in triaxial tests. Typical triaxial test results for varying joint orientations are shown below.

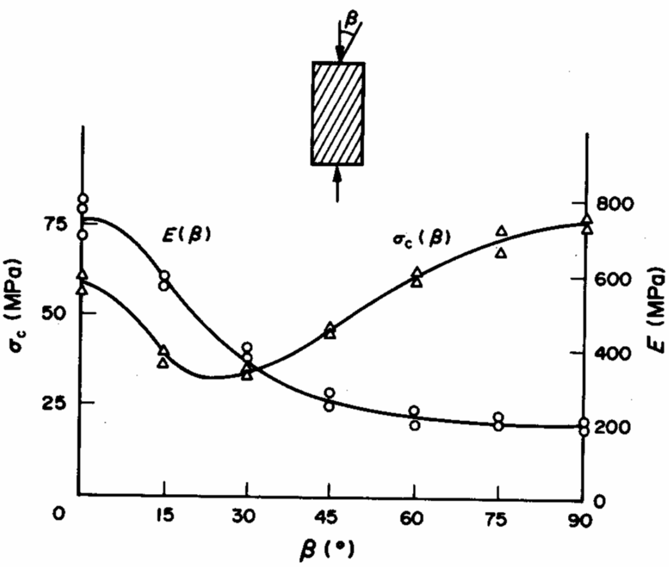


Figure 28: Rock strength and stiffness at varying joint orientations (Scott, 2007)

These results show a direct relationship between strength and stiffness at varying gradients consistent with the findings of Ramamurthy (2004) for joint orientations between 0° and the critical angle with respect to σ_1. In rock masses with joint orientations exceeding the critical angle, there appears to be an inverse relationship between rock strength and stiffness.

2. Fracture Characteristics

The roughness of rock surfaces has a profound impact on a variety of rock mechanics parameters such as, shear strength, rock stiffness and groundwater flow (Sakellariou et al., 1991).

Work by Demeris (1974), as well as Lama and Gonano (1976) showed that misfit between the blocks could give rise to stress gradients that result in reduced strength (Scott, 2007).

3. Fracture Frequency

With an increase in fracture frequency the interlocking of the rock fragments generally decreases (Smith, 2004).

Traditional mathematical models use the principle of superposition to predict the strength and stiffness of a rock mass for sliding failure modes. These results are only reasonable if the interaction between the joint sets can be ignored. A better understanding of the interaction between joint sets is required to clarify the complex behaviour of rock masses (Yang et al., 1998).

4. Water

Studies on the effect of water on the shear strength of rock joints by Barton (1973) showed that under low to medium stress levels:

- The shear strength of a planar surface is largely unaffected if wet; but
- The shear strength of rough joints reduces by between 5% and 30%.

It is thus considered that the presence of water does not significantly affect the shear strength of the discontinuity network.

In closely jointed rock masses, the effective stress is significantly influenced by the pore water pressure at lower stress levels, but becomes insignificant at higher stress levels where water tends to dissipate rapidly when loaded (Scott, 2007).

Thus, all the studies show that the reduced strength of the rock mass due to the presence of water is not more than the reduced strength of the intact rock. Within a rock mass, water does not reduce the strength, but rather acts as destabilising agent.

2.2.2 Weak rock mass under high confinement

Triaxial experiments by Ramamurthy and Arora (1994), as well as Tiwari and Rao (2006) showed that, with increasing confining pressure, the influence from the discontinuity network (including the effect of fracture orientation) becomes less significant (Wasantha, et al., 2014).

Under high confinement (deeper than ~ 1 km), rock mass behaviour is generally associated with rock material characteristics and the fracture network considerations may no longer be appropriate (Schultz, 1996). This is consistent with biaxial compression results by Muller and Pacher (1965) that showed for a principal stress ratio of $\sigma_1/\sigma_3 \geq 3$, the rock mass strength is almost equivalent to that of an intact rock sample.

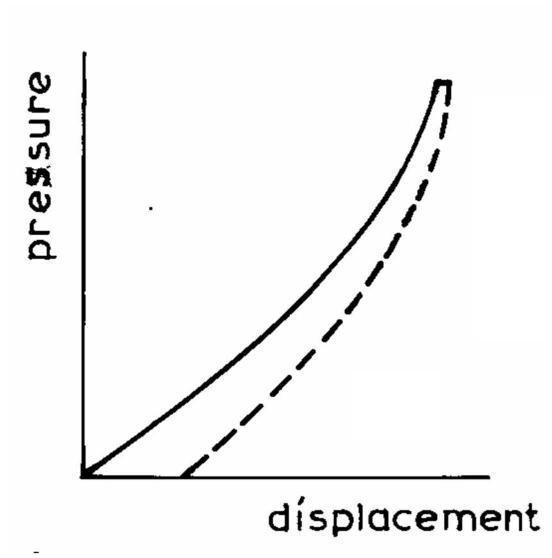
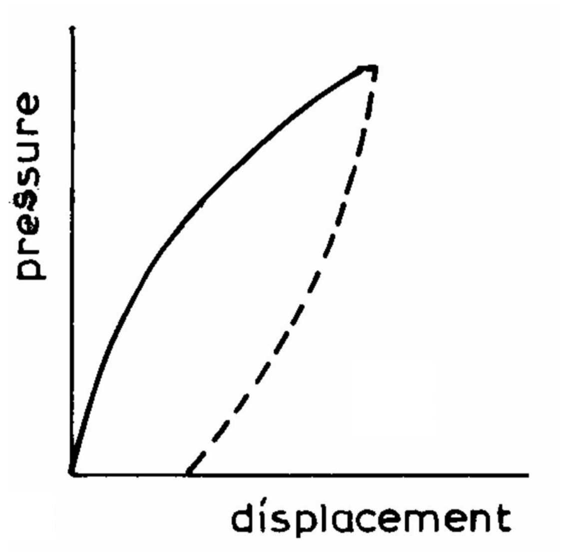
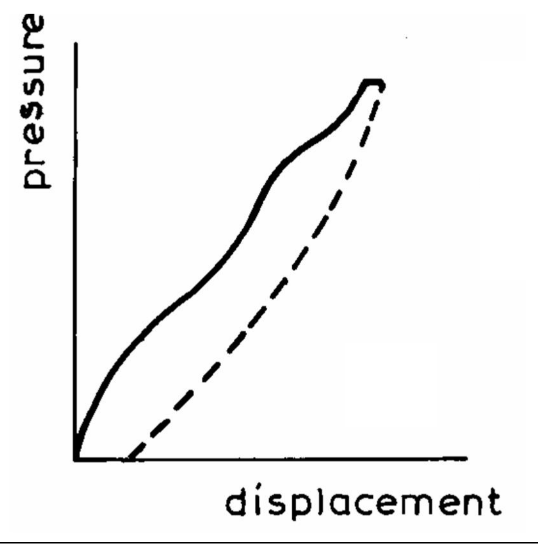
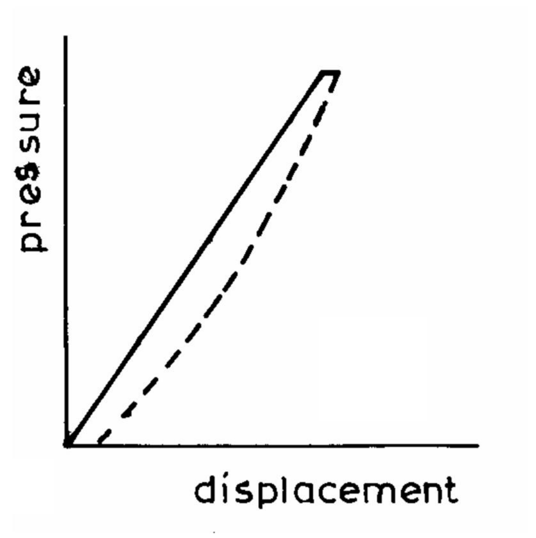
Rock mass under high confinement is thus expected to behave like soft rock as discussed in Section 2.1.

2.2.3 Direct testing methods for continuous rock masses

In an attempt to measure the characteristics of rock masses, traditionally the most general approach was to adapt known laboratory tests to a bigger scale in the field. Field tests are expensive and as they are not carried out within a controlled environment, there are additional interpretive difficulties (Scott, 2007).

Plate load test

The plate load test is one of the most documented field testing techniques. Traditionally, this rock stiffness testing technique measures the deformation under a loaded plate, but due to bedding errors and stress release of the rock upon excavation, it is found that the only way to obtain reasonable values for rock mass is with extensometers (Scott, 2007). Typical curves observed from plate load testing are shown in Figure 29:

<table>
<tr><td>
</td><td>
</td></tr>
<tr><td>Highly fractured hard rock mass</td><td>Soft rock mass</td></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td>Localised failures within the rock mass</td><td>Rock mass showing similar response to intact rock material (mass previously subjected to high stresses such as in deep excavations)</td></tr>
<tr><td colspan="2">Figure 29: Typical plate load results for rock masses (Scott, 2007)</td></tr>
</table>

Limitations:

- There is no indication as to the effect of joint orientations on these curves. It would therefore, not give reliable parameters other than for the tested direction. However, the curves do provide a general feeling for the expected behaviour of the rock mass.
- Scale dependence – Only suitable for very highly fractured rock.

Direct shear test

This technique measures the shear strength for a given normal stress. Like shear box tests in the lab, it is necessary to consider the rate at which the sample is sheared.

An analysis of several recorded direct shear tests revealed that the horizontal displacement at yield is generally between 15-30% of the horizontal displacement at failure (Scott, 2007).

Pressuremeter tests

These tests, which were originally developed for soil applications, are performed in boreholes, and assume a cylindrical cavity within a linearly elastic medium (Isik et al., 2008). Some researchers (like Haberfield and Johnston, 1990), however, argue that radial cracking occurs at a relatively early stage of expansion, affecting the stress-strain curve. The deformation modulus is thus affected by the cracking phenomena.

Additionally, care should be taken when the pressure test is in contact with soft seams as this significantly influences the results (Isik et al., 2008).

Geophysical surveys

Geophysical surveys used to estimate the rock stiffness, described in Section 2.1.4, assume a continuous, isotropic, homogeneous material. The effects of the discontinuity network on wave propagation is thus generally ignored.

Scott (2007) noted that, for closely jointed rock masses, shear wave velocities will tend to be low because of attenuation through the discontinuity network.

Stacey (1976) considered the effect of the discontinuity network in hard rock masses and found that the compressive wave velocity is generally insensitive to wet and clay filled joints, whereas the shear wave shows more promise. Experimental results show that the velocities of both the compressive and shear waves decrease with an increase in fracture frequency. The amount of decrease in the respective waves are, however, not the same. This is demonstrated in Figure 30 below.

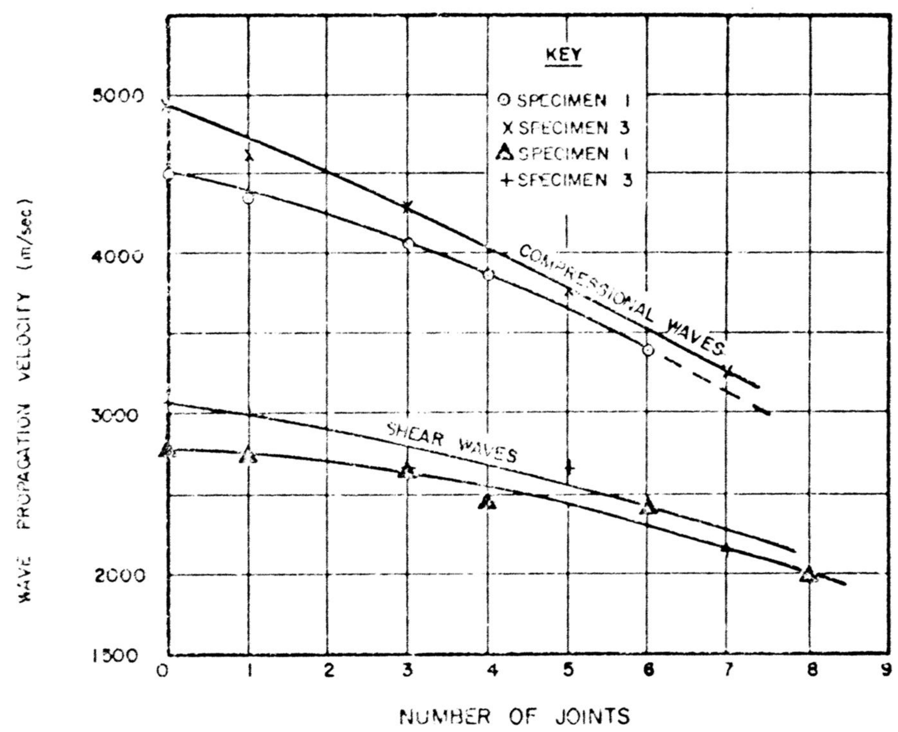


Figure 30: Effect of joint frequency on wave propagations under atmospheric pressure (Stacey, 1976)

These results show a direct relationship between fracture frequency and the time lapse between the first arrivals of the compressive and shear wave respectively. It is therefore plausible that the quality of the rock mass may be estimated through the difference between wave arrival times (i.e. the time laps between the two wave arrivals becomes less as the rock mass quality reduces). However, additional work will be required to assess this hypothesis.

Experimental studies also indicate that the extent to which wave propagation is affected by joint frequency is dependent on the joint orientation. This is illustrated in Figure 31.

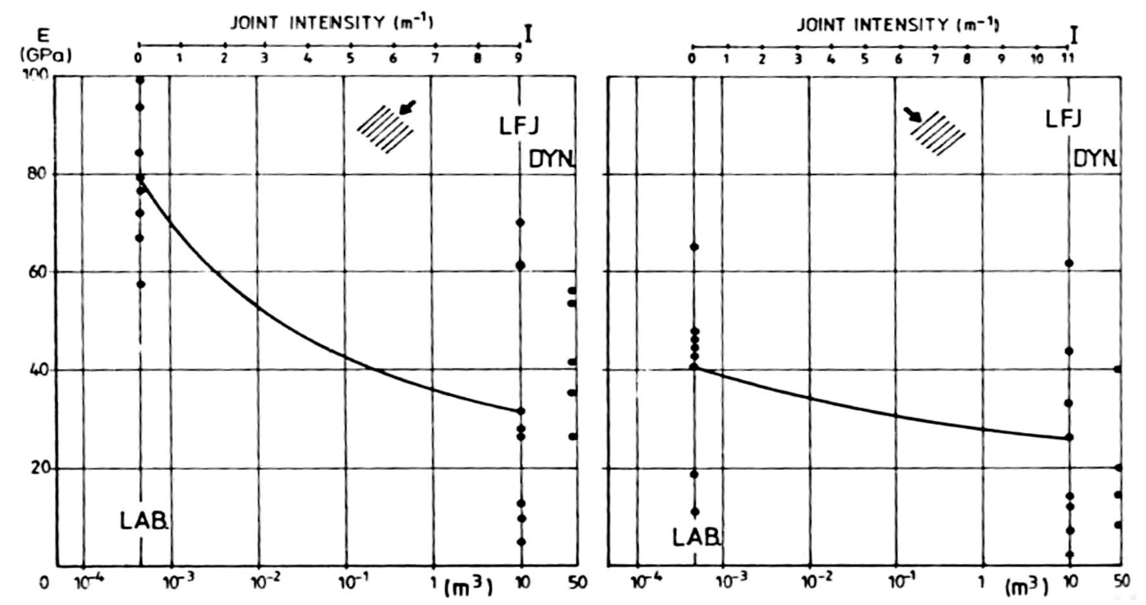


Figure 31: Effect of orientation on joint frequency during wave propagation (Pinto da Cunha & Muralha, 1990)

Additionally, the current model used to estimate static stiffness from shear wave velocity, assumes that the dynamic stiffness estimate represents the maximum stiffness estimate and the stiffness decreases with increasing strain. However, this is not always true for rock masses as the stiffness is expected to increase during the closing of joints when the rock mass is loaded (i.e. under a foundation) (Clayton and Heymann, 2001).

Available studies do however indicate that the effect of the discontinuity network is only applicable under low confining conditions. The confinement where the influence of the discontinuity network becomes insignificant appears to be related to rock strength, with estimates between 2MPa in hard rock masses and 0.4MPa in chalk (Stacey, 1976 and Clayton and Heymann, 2001).

He (1993) considered the effect of scale on various field tests and found that, although some variation does exist between the various methods, it is not as affected by scale compared to laboratory results.

2.2.4 *Indirect methods to estimate the strength and stiffness of continuous rock masses*

Generally, for rock masses, a reduced strength is assumed that is estimated from the degree of fracturing and the strength of the fracture network. These estimations are made with the aid of rock mass classification systems of which the Q, RMR and GSI systems are most commonly used for geotechnical engineering applications. Variables considered in each of these classification systems include:

- The Q system considers the degree of fracturing, the fracture network characteristics, ground water and the *stress environment*;
- RMR considers *intact rock strength*, degree of fracturing, ground water and the fracture network characteristics;
- GSI (which is based on RMR) considers degree of fracturing, the fracture network characteristics and *disturbance due to blasting*. Note that the blast damage is generally confined to the area in the vicinity of the face. The affected zone generally ranges from one or two metres for small civil engineering blasts (Hoek and Marinos, 2009).

These classification systems were developed for hard rock masses. When soft rock masses are considered, only the RMR consider the lower intact strength.

Additionally, the GSI system was later extended to include weak rock masses like foliated and sheared material (Hoek et al., 1998), as well as for heterogeneous rock masses, such as flysch (Marinos and Hoek, 2001). These extensions did, however, receive criticism, since the assumption on which the original system is based considered a homogeneous, isotropic medium.

Moon, et al. (2001) found that these rock mass classification systems gave poor slope angle estimates for weak rock masses. This may be due to failures through intact rock material, rather than solely along existing discontinuities (Douglas, 2002).

Various researchers have subsequently proposed extensions of known classification systems for weak rock masses. Two of these include (Russo, 1994):

- RMR System extension by Robertson (1988), for slopes; and
- MRMR System by Laubscher and Taylor (1976), for mining slopes.

Generally, rock mass classification systems are considered a practical application for a systematic evaluation of the rock mass quality, although they only give reasonable predictions within the set of applications for which the system was created and do not distinguish between different failure modes (Scott, 2007).

Empirical relationships were developed as extensions of strength criteria based on physical model results, small scale laboratory tests and limited experience, in order to estimate the strength and stiffness of rock masses (Douglas, 2002).

These estimates are used in the absence of reliable in-situ data for preliminary designs (Bieniawski, 2011). One of these correlations is shown in the Figure below. Other documented correlations are included in Table 1.

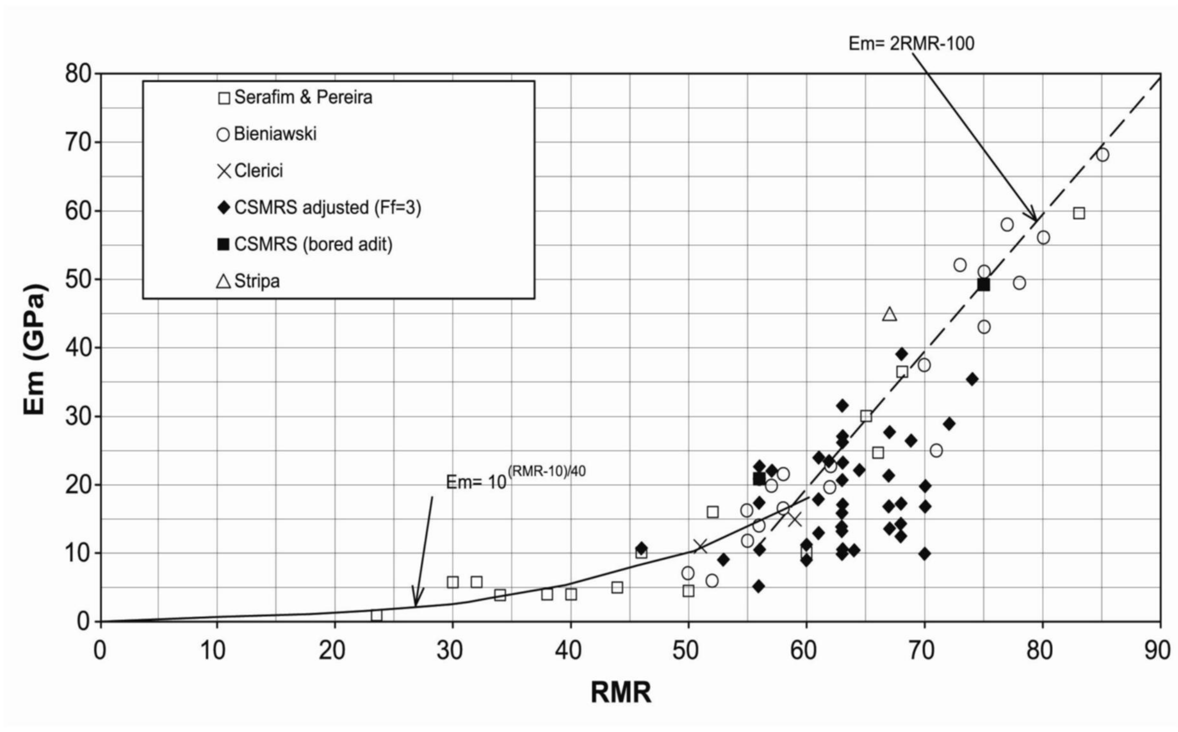


Figure 32: Rock mass stiffness estimation based on RMR classification system (Palmström & Singh 2001)

Table 1: Available Empirical Correlations	
Correlation	***Reference***
$\frac{UCS_{RM}}{UCS_i} = \exp\left(7.65\frac{RMR-100}{100}\right)$	Yudhbir and Prinzl, 1983 RMR_{76} Version*
$\frac{UCS_{RM}}{UCS_i} = exp\left(\frac{RMR-100}{18.5}\right)$	Ramamuthy, 1986
$\frac{UCS_{RM}}{UCS_i} = exp\left(\frac{RMR-100}{20}\right)$	Sheorey, 1997
$E_{RM} = E_i\left[0.0028RMR^2 + 0.9exp\left(\frac{RMR}{22.82}\right)\right]/100$	Nicholson & Bieniawski, 1990
$E_{RM} = 300\exp(0.07RMR)\times 10^{-3}$	Kim,1993
$E_{RM} = \frac{E_i}{2}\left[1 - cos\left(\pi\times\frac{RMR}{100}\right)\right]$	Mitri, et al., 1994
$\frac{UCS_{RM}}{UCS_i} = exp\left(\frac{RMR-100}{24}\right)$	Kalamaris & Bieniawski, 1995
$\frac{UCS_{RM}}{UCS_i} = \left(\frac{RMR}{RMR + 6(100 - RMR)}\right)$	Aydan & Dalgic, 1998
$E_{RM} = \sqrt{\frac{UCS}{100}}10^{(GSI-10)/40}$	Hoek and Brown, 1997
$E_{RM} = 10\left(Q\frac{UCS}{100}\right)^{1/3}$	Barton, 2002
$E_{RM} = E_i 10^{\left[\frac{(RMR-100)(100-RMR)}{4000\exp\left(-\frac{RMR}{100}\right)}\right]}$	Sonmez, et al., 2006
$E_{RM}[GPa] = \frac{100}{1 + e^{\left(\frac{75-GSI}{11}\right)}}$	Hoek and Diederichs, 2006

The relationship proposed by Yudhbir, et al (1983), Ramamuthy (1986), Sheorey (1997), as well as Kalamaris & Bieniawski (1995) assume a non-zero unconfined compressive strength (UCS) and hence tensile strength, for the rock mass. These criteria could therefore over predict the strength for poor quality rock masses under low confinement (Douglas, 2002).

Bieniawski (1989) found that known empirical relationships for the deformation modulus based on rock classification systems give reasonable predictions for field conditions, however, this is only valid for high-modulus rock types (E > 10 GPa) (Schultz, 1996).

In order to assess the applicability of the various documented correlations, a comparison was done on four UCM test results, two samples with a high UCS value (hard rock) and two soft rock samples respectively.

The results are shown in Figure 33 and Figure 34 respectively.

Assumptions:

- The tested E_{tan} modulus was considered the intact rock material stiffness at 0.1% strain, which correspond to foundation loading according to Mair et. al. (1993) (Refer to Figure 20);
- A corresponding rock stiffness at 1.0% strain, which corresponds to the max strain during tunnelling according to Figure 20 was estimated in accordance with Figure 19;
- The resultant strengths were multiplied by an average to low modulus ratio of 200 and the stiffness estimates used in the comparison; and
- The results of all four samples were considered to evaluate the strain level for each of the empirical relationships.

Findings from the comparison:

- Empirical relationships by Ramamuthy (1986), Kalamaris & Bieniawski (1995), Sheorey (1997), Hoek and Brown (1997) and Palmström & Singh (2001), consider stiffer material at lower strain levels that is more appropriate for foundation design and slope stability analysis;
- Stiffness estimates at lower strain levels are susceptible to significant scatter. One of the reasons may be the effect of joint orientation that is not assessed within the known correlations;

- A comparison between the empirical relationship proposed by Palmström & Singh (2001) that only considers the RMR value and that of Hoek and Brown, which also considers intact rock strength, indicate stiffer estimates for hard rock masses and a lower stiffness for soft rock masses when the Hoek and Brown relationship is used. The Hoek and Brown relationship is considered to best describe rock mass stiffness in slopes and it provides more reasonable estimations when soft rock masses are considered;
- Empirical relationships by Nicholson & Bieniawski (1990), Mitri, et al. (1994), and Barton (2002), consider softer material at higher strain levels that is more appropriate for a rock mass behavioural analysis in tunnels.
- There is less scatter associated with stiffness estimates obtained at a higher strain level.

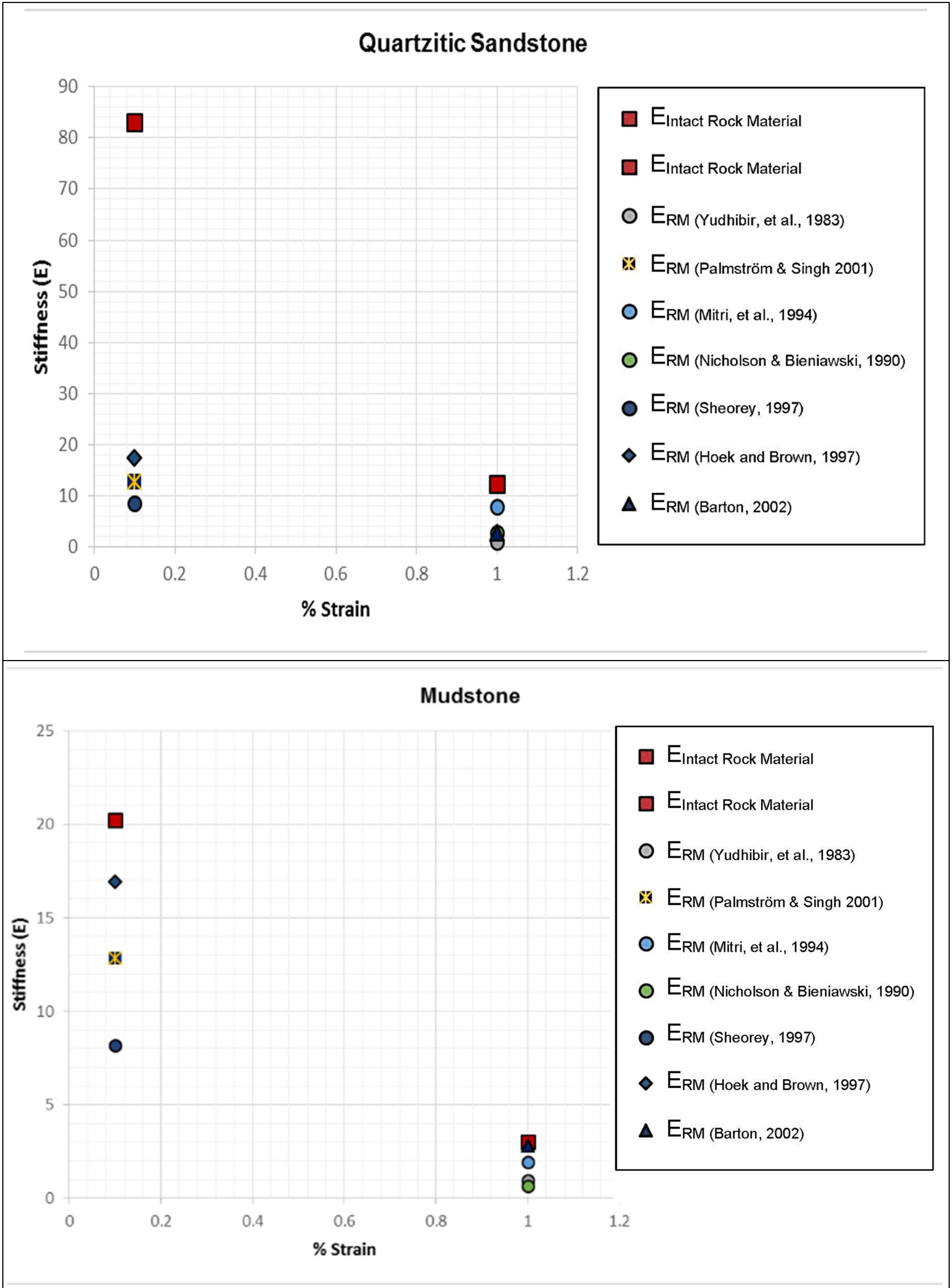
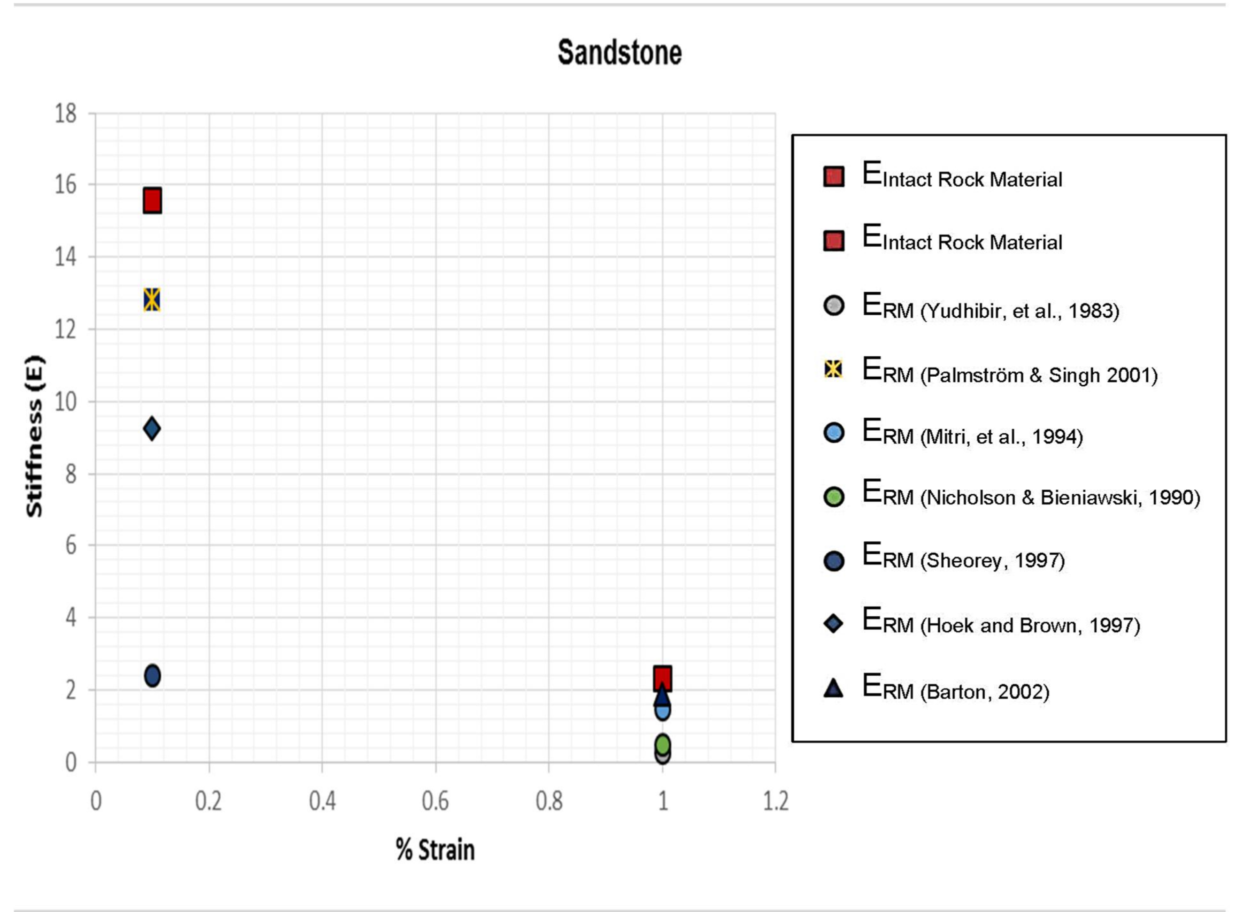


Figure 33: Comparison between known empirical relationships in hard rock mass



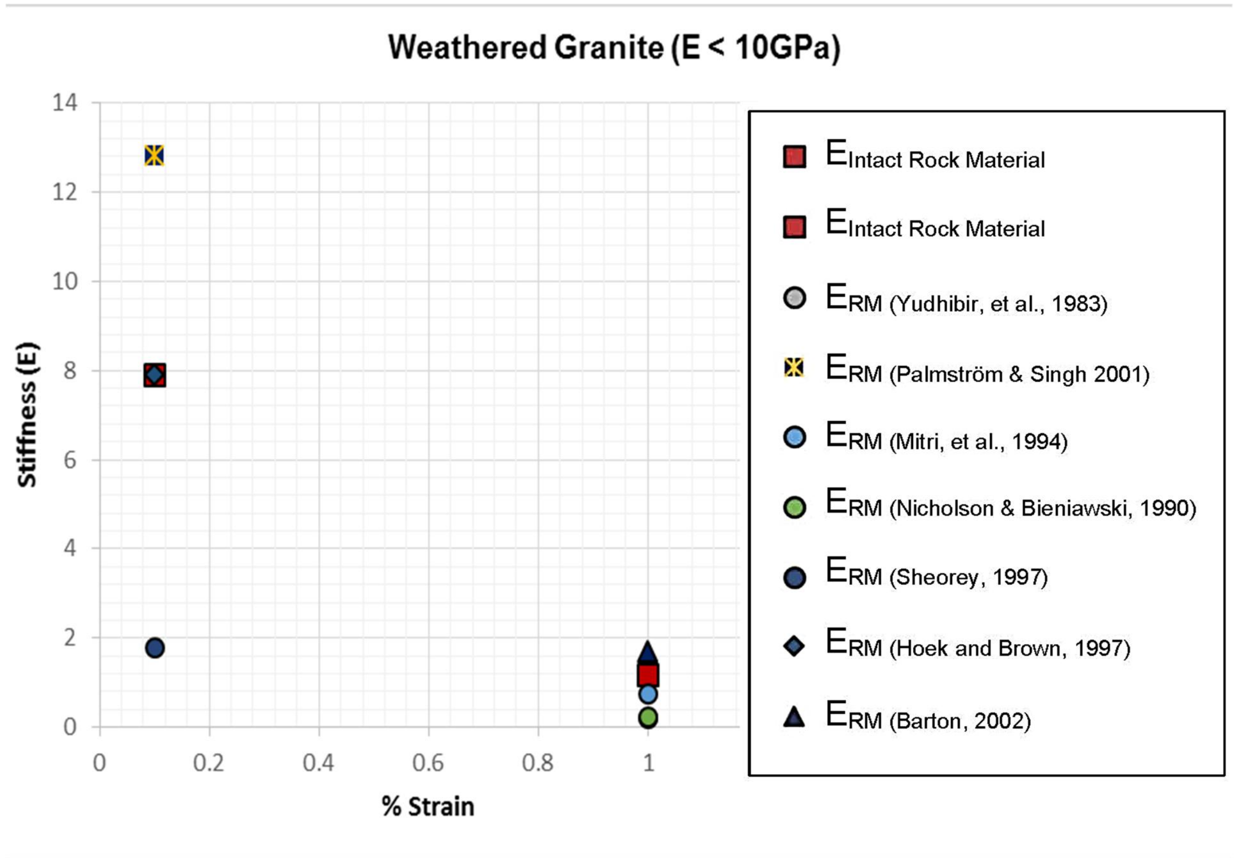


Figure 34: Comparison between known empirical relationships in soft rock mass

2.3 Bimrocks

Within nature, there are variable masses that are characterised by hard fragments in a soft matrix. These rock masses are associated with certain geological settings (i.e. conglomerate, or melange) or due to weathering and are collectively called bimrocks. In bimrocks, the shear strength of the mass is greater than the shear strength of the matrix due to a tortuous failure path around the rock fragments.

An increased strength is only possible when there is a sufficient contrast between the shear strength of the blocks and matrix respectively. Material with UCS_{block}/UCS_{matrix} >1.5 is considered bimrock (Medley and Zekkos, 2011).

The shear strength of the rock mass is dependent on:

1. *The proportion of blocks*
   - When the proportion of hard blocks is less than 25 %, the strength of the rock mass is generally similar to the strength of the rock matrix (Santi, 2006);
   - Work by Santos et al. (2002) on gravel in clayey soil indicated an increase of the internal friction angle at a rate of about 2.5° for each additional 10% of gravel. The shape and orientation of the gravel proportion, which has been found to influence the rate at which the friction angle increases, is not considered in this analysis;
   - The increase in the overall frictional strength can be up to between 15° and 20° above the matrix friction strength (Medley and Sanz Rehermann, 2004). Additionally, there appears to be no additional increase in friction angles when the block proportion exceeds 70 % (Medley and Zekkos, 2011).
   - Results of physical models by Lindquist and Goodman (1994) indicated a decreased cohesive strength in relation to the increase in internal friction angle (Haneberg, 2004). An estimate of the associated loss of cohesion with an increase in block proportion, is however not found in the available literature.

2. *Block orientation*
   - Studies by Lindquist and Goodman (1994) showed that the friction angle and modulus of deformation is dependent on the relative orientation of blocks to applied stresses. When blocks are orientated at orientation of about 30°, the lowest strength estimate is obtained (Kim et al., 2004).
   - Blocks that generally dip out of the slope decrease the slope stability, but blocks that are oriented at high angles to the slope increase the slope stability (Medley and Sanz Rehermann, 2004).

3. *Block size distribution*

   Bimrocks that are characterised by uniformly sized blocks generally have a lower rate of shear strength increase, compared to a bimrock with a varying block distribution (Medley and Zekkos, 2011).

4. *Block shape*

   An estimate of the effective shear strength of the bimrock is strongly dependent on the block proportion in the material. There is however significant uncertainty associated with block proportion estimates based on 1D (borehole logging) and 2D (face mapping) sampling techniques (Haneberg, 2004).

   A reasonable estimation of the 3D volumetric block proportion is commonly influenced by, amongst others, the amount of sampling, block shape, block size distribution and the orientation of the blocks.

   Should the blocks be roughly equal-dimensional, the volumetric block proportions in 3D could be assumed to be the same as the block proportions obtained from 2D measurements (Sönmez et al., 2004).

   When the bimrock is characterised by ellipsoidal blocks, practitioners commonly deal with the uncertainty by means of statistical techniques, assuming a normal distribution of apparent block sizes within a mass.

Bimrocks are scale independent, with a blocky nature found at all scales of engineering interest (Medley and Sanz Rehermann, 2004). Thus, in contrast to soft rock and rock masses that are considered in the previous sections, laboratory scale samples may be representative of site scale rock behaviour (Medley and Zekkos, 2011).

Additional studies were carried out to compare the rock strength to fracture patterns. The results (some of which are indicated in the Figure below) revealed little systematic variation between the geometry of failure surfaces with block proportions and block orientations (Medley and Sanz Rehermann, 2004).

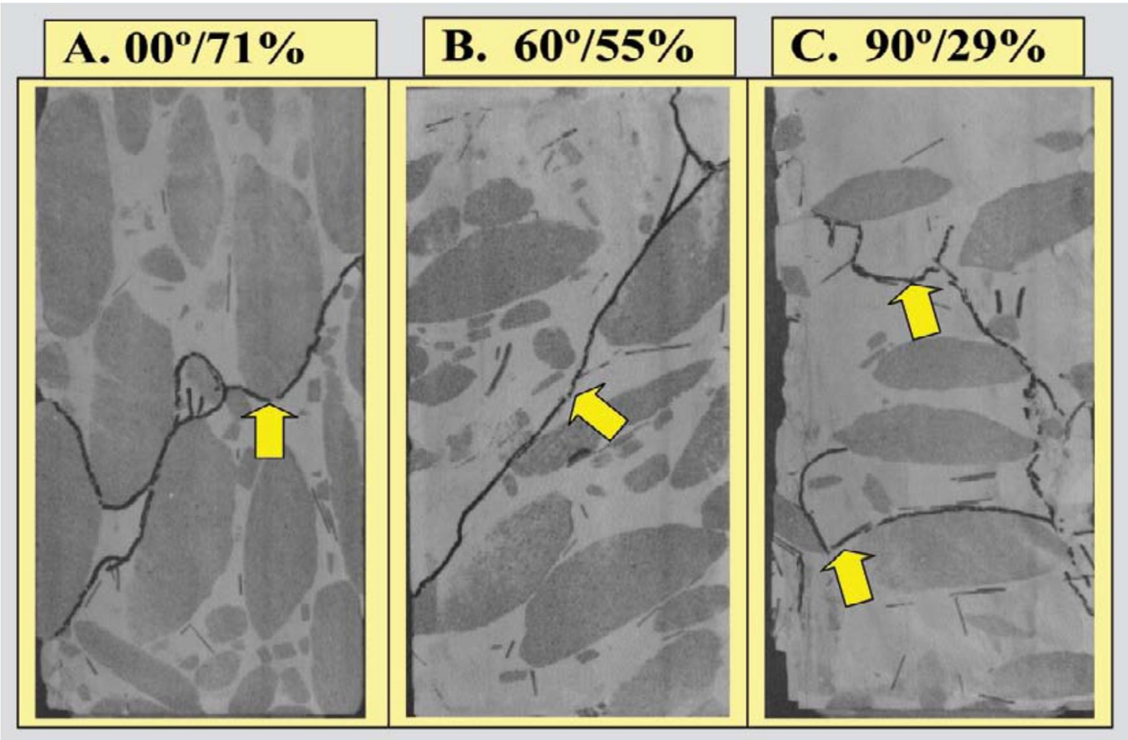


Figure 35: Effect of block orientation and proportion on fracture propagation (Medley and Sanz Rehermann, 2004)

Measurements from this analysis did however show that internal friction angle estimates based on conventional rock engineering approaches, which incorporate joint roughness coefficients (JRC), will be inappropriate, since the asperities of the failure surfaces in Bimrocks is significantly larger than the roughness estimates within the JRC standard profiles (Medley and Sanz Rehermann, 2004).

The mean width of the failure zone was found to be approximately 10 % of the sample diameter, with a standard deviation of 5 %. Considering the scale-independent nature of bimrocks, a 10 % thick failure zone at the scale of engineering interest could be assumed. Within slope stability studies, the scales of engineering interest are, appropriately, the slope heights (Medley and Sanz Rehermann, 2004). Failure zones associated with bimrocks are thus characterised by relatively thick zones of rock/soil mixtures (Medley and Zekkos, 2011).

Additionally, a preliminary study by Sönmez, et al. (2004) revealed an apparent relationship between the overall strength of the bimrock and the UCS of the blocks. The general assumption that overall bimrock strength is not influenced by block strength, as long as there is sufficient contrast between the block and matrix strength to force failure surfaces around blocks, may thus be an oversimplification. This concern may be particularly relevant under high confining pressures. Medley and Zekkos (2011) noted that under increased confinement, failure surfaces will penetrate blocks regardless of the difference between the shear strength of the blocks and matrix respectively.

2.4 Tools for material strength and stiffness determination / estimation

Variable strength and stiffness estimates are derived from different techniques. The choice of technique is usually controlled by in-situ conditions, availability of testing technique and budgetary constraints. Available techniques to obtain parameters that define the behaviour of weak rock masses include:

- *Laboratory tests*
  Laboratory techniques are limited to small samples and generally provide estimates for the *intact rock material*, which is known to overestimate the material strength in the field by up to 30%.
- *Field tests*
  Boundary conditions are often not known, which creates a degree of uncertainty in the interpretation of the results.
- *Empirical formulae*
  Only used in the absence of in-situ data for preliminary designs.

With reference to Figure 2, available techniques to estimate the strength and stiffness of intact soft material include:

Rock material strength

Laboratory tests are conducted on rotary core samples. When the core is recovered in sections of significant length, UCS tests are carried out. *Studies did, however, indicate that UCS measurements on soft rock samples, tested under standardised loading rates, are generally grossly overestimated.*

When the core is fractured, it may become difficult to impossible to obtain a suitable sample for UCS tests. Correlations with indirect testing methods are then used. These correlations have an intrinsic degree of uncertainty due to significant scatter in the results from alternative techniques. Alternative indirect testing methods include:

- Point load test

  In addition to the natural scatter associated with this technique, Kanji (2014) also recognised that soft rock samples generally deform significantly before failure, which may affect the measurement.

  An evaluation on proposed correlations, presented in Figure 8 showed, that contrary to the general belief that the correlation is dependent on the rock strength, the correlation appears to be related to rock composition. The recommended correlations based on the analyses includes:

  - $UCS_{(Crystalline\ rock)} = 24 \times I_{s(50)}$,
  - $UCS_{(Sandstone)} = 14 \times I_{s(50)}$, and
  - It is considered that there is no real correlation for mudstone or shale and should thus not be used for these materials.

- Schmidt Hammer

  The Schmidt hammer comprises of a portable device that measures the rebound hardness of the rock (Ulusay 2015). One of several correlations between the rebound hardness and UCS is included as Appendix A in this report.

  Measurements with the Schmidt Hammer can, however, be affected by neighbouring fractures within the rock outcrop (Mol, 2016).

When rock is too soft for UCS tests:

- Triaxial test
  This test is time consuming and relatively expensive, but gives reasonable strength and stiffness estimates.

<u>Rock material stiffness:</u>

When the core is recovered in sections of significant length, rock stiffness can be measured with additional strain gauges during a UCS test. This is commercially known as a UCM test.

When the core is very soft or fractured, correlations with indirect testing methods have been derived:

- Bender elements in very soft rock
  In this technique, the elements create a seismic wave that is measured in the lab. This technique is, however, not commonly available.

- Seismic surveys or CSW Tests
  The shear wave velocity is correlated with the rock stiffness, according to:
  - $G_0 = \rho V_s^2$

- Pressuremeter test
  This is an in-situ test that is carried out in boreholes. Limitations of this technique include:
  - Radial cracking affects the stiffness estimate, and
  - The test results are sensitive to the presence of soft seams in the area tested.

- Plate load test
  This is the most documented field technique, that has been used extensively for large projects throughout the world. This technique was presented in Section 2.2.3.

Rock Durability

Rock material loses strength upon weathering. Some types of rocks have slake durability issues, resulting in the disintegration of the rock material in the design life of the structure due to exposure to water or climate change. Techniques used to verify the long-term characteristics include:

- Mineralogical analysis
  Indications of material prone to slaking:
  - Mudstones with less than 56% quartz;
  - Rock with more than 10% Smectite;
  - The presence of deleterious elements (i.e. iron sulphides with chlorite and sodium).

- Wetting and Drying
  Use Nickmann, et al.'s (2006) classification system to assess potential weathering rate:
  - Spontaneous decay – Less than 25% of the mass remaining after the 1st Cycle;
  - Rapid decay (days) – 25 to 90% of the mass remaining after the 1st Cycle;
  - Moderate decay (weeks) – More than 90% of the mass remaining after the 1st Cycle, but less than 50% of the mass remaining after the 3rd Cycle; and
  - Slow decay (months to years) – More than 90% of the mass remaining after the 1st Cycle, with 50 - 95% of the mass remaining after the 3rd Cycle.

- Slake durability test

Strength estimation of the fracture network

The shear strength of a joint plane is estimated through the basic friction angle that considers a minimum shear resistance on a smooth, planar surface in fresh rock. The basic friction angle may be considered a material constant (Kemthong, 2006).

Studies by Kemthong (2006) showed a weak correlation between UCS and the basic friction angle, which seem to be dependent on rock type.

This relationship is explored further with laboratory test results from the N2 Wild Coast Project. Here the relationship between UCS and shear box test results on samples within 2m along the same core length were considered. The result of this analysis is shown in Figure 36.

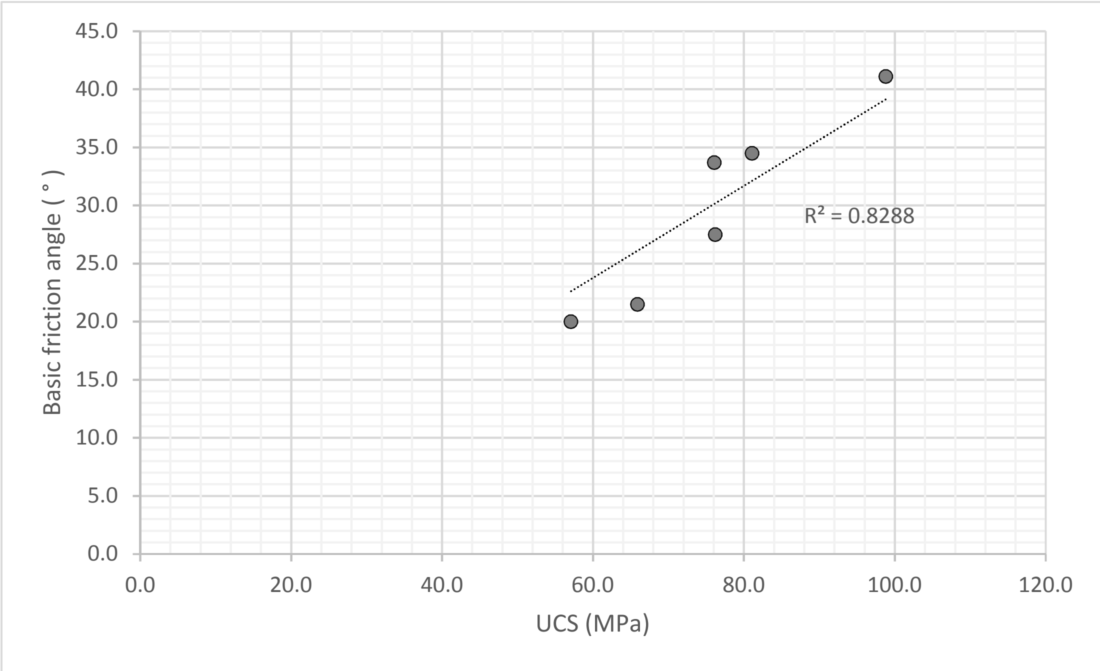


Figure 36: Correlation between UCS and basic friction angle for sandstone

In accordance with previous findings, it appears that there is a reasonable correlation between UCS values and the basic friction angle and thus also an apparent correlation between the compressive and shear strength of a rock material. The correlation given as Equation 5 is thus a reasonable assumption.

Available methods to test the shear strength include:

- Shear box test in the lab; and
- Direct shear test in the field.
  The direct shear test is expensive and only used when there are weak planes in a hard rock mass that is critical for the stability analysis. Here, the basic

friction angle is not suitable as it would overestimate the shear strength of the rock mass (Kemthong, 2006).

Rock mass strength estimation

A reduced strength for the mass is estimated by rock mass classification systems.

Available systems that are most commonly used for geotechnical engineering applications include the Q, RMR and GSI classification systems that permit an evaluation of the reduction in the quality of the rock mass due to fracturing.

Available empirical correlations to estimate rock mass strength and stiffness are given as Table 1 in Section 2.2.4.

Rock mass stiffness

In addition to the empirical correlations, a number of field tests have been developed to measure the rock mass stiffness, the most commonly used technique being the Plate Load Test. Various researchers have, however, shown that reasonable stiffness measurements with the plate load test can only be obtained using extensometers.

Bimrocks

In this group of materials, the strength of the blocks and the strength of the matrix are tested using techniques described above for the rock material. The strength of the mass is then evaluated through the block proportion at the scale of interest. The recommended procedure by Medley and Zekkos (2011), is shown in Figure 37.

STEP 1

- Determine block matrix contrast and evaluate it by:
  $\tan\phi_{block}/\tan\phi_{matrix} \geq 1.5–2.0$
  $E_{block}/E_{matrix} \geq 2.0$
  $UCS_{block}/UCS_{matrix} \geq 1.5$

STEP 2

- Select characteristic dimension (L_0) at the scale of interest (ie slope height or width of tunnel face).

STEP 3

- Calculate block size threshold
  $d_{min}=0.05*L_c$ (block-matrix threshold)
  $d_{max}=0.75*L_c$ (block–blocky rock mass threshold)

STEP 4

- Schedule drilling with minimal hole length = 10 x d_{max}

STEP 5

- Measure total linear block proportion B_L

STEP 6

- Assume B_L = Volumetric Block Proportion (B_V), and estimate uncertainty range for this assumption

STEP 7

- Design conservatively, using lower bound for strength estimation and upper bound for earthwork construction.

Figure 37: Seven golden steps to estimate bimrock strength (Medley and Zekkos, 2011)

Global advances have been made in testing techniques, one the main ones being down the hole triaxial tests (Taheri and Tani, 2008). Additional work is still required to produce standardised methodologies and more affordable “off the shelf” equipment. This technique is not yet locally available in South Africa.

2.5 Rock Strength and Stiffness Evaluation for Design

2.5.1 Available techniques for analysis

Stress Criterion

The behaviour of the rock (mass) is affected by the net distribution of stresses on the material. Currently, within continuum mechanics, it is generally accepted that the rock (mass) strength refers to the predicted yield stress and failure stress for a given design (Douglas, 2002).

Within practical applications, rock strengths in varying stress conditions are estimated through theoretical failure criteria. An understanding of what the estimate represents and inherent assumption of analytical tools that predict the strength for a specified stress environment is thus crucial for a reasonable design.

Some of the most widely referenced criteria are given in Table 2.

Table 2: Most widely referenced strength criteria

Author	*Criteria (formulation)*	*Recommended Classification*	*Comments*
Mohr-Coulomb Criterion by Holtz and Kovacs. (1981)	$\tau = c + \sigma_n \tan\varphi$	Based on triaxial test results	A linear strength criterion based on the work of Otto Mohr and Charles-Augustin de Coulomb
Hoek & Brown (1980)	$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$	RMR_{76}; GIS_{2002}	Parameter m_b appears not to be related to rock type alone, but rather a rock type under specified confining pressures.
Bieniawski (1974) Yudhbir, et al. (1983)	$\sigma_1 = A\sigma_{ci} + B\sigma_{ci}(\frac{\sigma_3}{\sigma_{ci}})^\alpha$	RMR_{76}; Q	This criterion does not consider tensile strength, which implies a friction angle of 90° at $\sigma_3 = 0$.
Sheorey, et al. (1989)	$\sigma_1 = \sigma_{cm}\left(1 + \frac{\sigma_3}{\sigma_{tm}}\right)^{b_m}$	RMR_{76}; Q	This criterion does not allow for a tensile strength of zero.
You (2009)	$\sigma_S - \sigma_3 = Q_\infty - (Q_\infty - Q_0)e^{\left[-\frac{(K_0-1)\sigma_3}{Q\infty-Q0}\right]}$	Based on triaxial test results	This criterion over predicts tensile strength and a cut-off is recommended (You, 2015)

However, these criteria do not always predict the failure strength accurately. Some of the explanations found for unreasonable estimates include:

- The criterion's inherent assumption of a 2D, isotropic medium, making it impossible to vary the stress orientation on the critical failure plane in the rock mass (Douglas, 2002).
- Ma, et al. (2014) showed that the measured strength is larger when $\sigma_2=\sigma_3$ compared to $\sigma_2=\sigma_1$. Higher values where $\sigma_2=\sigma_3$ are usually measured commercially with laboratory apparatus. This effect is more significant in hard brittle rock and thus gives reasonable predictions for soft rock (masses).
- These criteria assume that the cohesion and internal friction angle are both contributing to the rock (mass) strength simultaneously, but this is not always the case. Sometimes the rock fails when the cohesion is exceeded, and friction builds up as the mass deforms.

Criteria that are predominantly used in geotechnical assessments include Mohr-Coulomb (linear) and Hoek-Brown (non-linear), both involving stress based *failure of brittle rock under confining conditions*, and are usually included in commercial software packages. These criteria consider elastic conditions up to failure, with values below $\sigma_1=3.4\sigma_3$, which is slightly different to the brittle ductile transition suggested by Mogi (1966), where $(\sigma_1- \sigma_3) = 3.4\sigma_3$. During the development of these criteria, the tensile strength is not defined, but is considered to be 1/10th the UCS value (Bieniawski, 1974). Experimental work by Hoek and Martin (2014), however, showed the stress for tensile crack initiation is roughly 0.45 times the UCS.

Since rock and rock masses are known to behave in a non-linear manner, a non-linear criterion like the Hoek Brown criterion is usually preferred.

Some experiences with the Hoek-Brown criterion on weak rock masses include:

- Analysis of Melbourne mudstone shows that this criterion generally provides poor estimations for soft rock material, mainly due to the fixed exponent that restricts the curvature of the envelope. Where the intact rock comprises a soft rock, the curvature is much less pronounced than predicted (Johnston & Chiu, 1984).

- There is a poor correlation between m_i and rock types, found in published tables. Experimental work did however indicate a close correlation between m_i and the ratio of σ_c (UCS)/σ_t (tensile strength) (Douglas, 2002).

- The Hoek-Brown criterion estimates zero cohesion for intensely fractured rock masses. This is perhaps why it will under-predict the shear strength of cohesive masses at low confining pressures.

- On the other hand, Scott (2007) noted that the Hoek-Brown failure criterion overestimates the strength of an unweathered, intensely fractured greywacke at low to moderate confining pressures (he assumed this is due to *high* intact strength combined with intense fracture network).

Douglas (2002) thus proposed a modified Hoek Brown criterion for soft rock masses. The proposed modifications are included in Table 3.

Table 3: Modified Hoek-Brown Criterion (Douglas, 2002)

Parameter	*Original Criterion*		*Modified Criterion*	
	Intact rock	*Rock mass*	*Intact rock*	*Rock mass*
Exponent (α)	Fixed at 0.5	Based on formulae; $\propto = \frac{1}{2} + \frac{1}{6}(e^{-\frac{GSI}{50}} - e^{-\frac{20}{3}})$ (max 0.62 for GSI = 5)	$\alpha_i = 0.4 + \frac{1.2}{1 + exp\left(\frac{m_i}{7}\right)}$	$\alpha_{rm} = \alpha_i + (0.9 - \alpha_i)exp\left(\frac{75 - 30m_{rm}}{mi}\right)$ Varies up to 0.9 as m_{rm} approaches 2.5
s	1	$s = \exp(\frac{(GSI - 100)}{9})$	1	$s_{rm} = min\begin{cases} exp\left(\begin{matrix} GSI - 85 \\ 15 \end{matrix}\right) \\ 1 \end{cases}$ When there is sufficient clay (no rock to rock contact) the shear strength of the soil used
m	Based on rock type found in published table	$m_{rm} = m_i exp\left(\frac{(GSI - 100)}{28}\right)$	Estimated in triaxial tests using correlation; $m_i = \left\lvert\frac{\sigma_c}{\sigma_t}\right\rvert$	$m_{rm} = max\begin{cases} m_i \frac{GIS}{100} \\ 2.5 \end{cases}$

*GSI could be replased by RMR value

Results from numerous high pressure triaxial test results suggested a critical state when $\sigma_1 = 3\sigma_3$. Additionally, Singh, et al. (2011) noticed that the unconfined strength (σ_c) is roughly equal to σ_3 for all rock types. The complete shear strength envelopes of rock can thus be described based on the UCS test results. This simplification does not apply to the Mohr Coulomb or the Hoek-Brown criteria, where triaxial tests are required over a wide range of confining stress, to correct the envelope over a given stress range (Barton, 2016).

However, all of the stress based failure criterion considered above does not allow for tensile crack formation and propagation. Thus, these criterion does not consider all stress environments.

An example for conditions where the stress criterion will give unreasonable estimates include rock conditioning. Here the rock is fractured with fluid and as the fluid dissipates, the stress refers back to the initial condition (thus the original strength), but the newly fractured rock has a reduced strength.

Strain based criterion has subsequently been developed to estimate rock strength subjected to tensile crack formation.

<u>Strain Criterion</u>

Stacey (1981) proposed a three-dimensional extension strain criterion to describe the mechanism of sidewall slabbing. According to this criterion brittle rock start to form fractures when the total extension strain in the rocks reaches a critical value and the rock starts to dilate. Through experimental test results for several rock types, Stacey determined the critical values under uniaxial compression conditions, occurring at a stress level of approximately 30% of the ultimate strength. However, this criterion does not have a universal character and is only applicable to brittle rocks under low confinement. (Kwasniewski and Takahashi, 2010). The criterion is expressed as (Stacey, 1981):

$$\varepsilon_3 = \frac{1}{E}[\sigma_3 - v(\sigma_1 + \sigma_2)]$$ **Equation 12**

Where

- ε_3 is the minimal principal strain;
- E is the Elastic modulus; and
- v is the Poisson's ratio.

Subsequently, additional criteria were developed, including:

- Sakurai’s critical strain criteria (also modified and extended in 1995), where the maximum principal strain (ε_1) is derived from displacement measurements taken near an excavation and compared with the allowable value or the so-called critical strain ε_0 which can be determined from a ratio of the uniaxial compressive strength and the initial modulus of elasticity.

  Sakurai found that the critical strain assumed values ranging from 0.1 to 1.0% for rocks and from 1.0 to 5.0% for soils (Nyungu, 2013).

- Fujii’s critical tensile strain criterion.

  This comprise a criterion for brittle failure of the rock material where the minimum principal $\text{strain}_{(\text{extension})}$ is equal to a critical tensile strain. The critical strain is dependent on both rock strength and rock type, with suggested values (tabulated in the literature) based on laboratory test results (Kwasniewski and Takahashi, 2010).

  Experimental work by Fujii, et al. (1994), revealed that the circumferential strain is independent of the strain rate and confining pressure, but is to some degree influenced by the water content (Nyungu, 2013).

CHAPTER 3 - EXPERIMENTAL WORK

The previous Chapter focussed on a review of published information on the definition of weak rock masses and current practices for the estimation of the design parameters for this material.

In this Chapter some experimental work, carried out specifically for this Research Report, is described.

3.1 Introduction

Most of the available testing techniques has been developed to measure the compressional strength of intact rock. Although it is possible to estimate the shear strength of the rock through existing failure criteria, it is often very difficult to measure the shear strength of the intact rock with existing testing techniques. Available tests include the shear box, which was developed to measure the shear strength along discontinuities. The shear strength of intact rock is estimated from a measured basic friction angle.

There is thus a need for a quick, inexpensive test to measure the shear strength of intact rock.

Wiid (1981) offered an alternative technique, ideal for very soft to soft rock, in the form of a modified vane shear test. This potentially inexpensive alternative is ideal when sampling of soft rock without significant disturbance becomes very difficult or unlikely.

Through experimental work, Wiid (1981) showed a direct correlation between the measured strength of the vane shear test and the direct shear test. He even went further to investigate the influence of the length of the vane embedment to the measured results, and found that, with an increase in vane diameter, the deeper the vane should be embedded to obtain reasonable results. Based on these preliminary studies, the depth of embedment should be at least four times the vane diameter. Wiid's findings are shown in Figure 38.

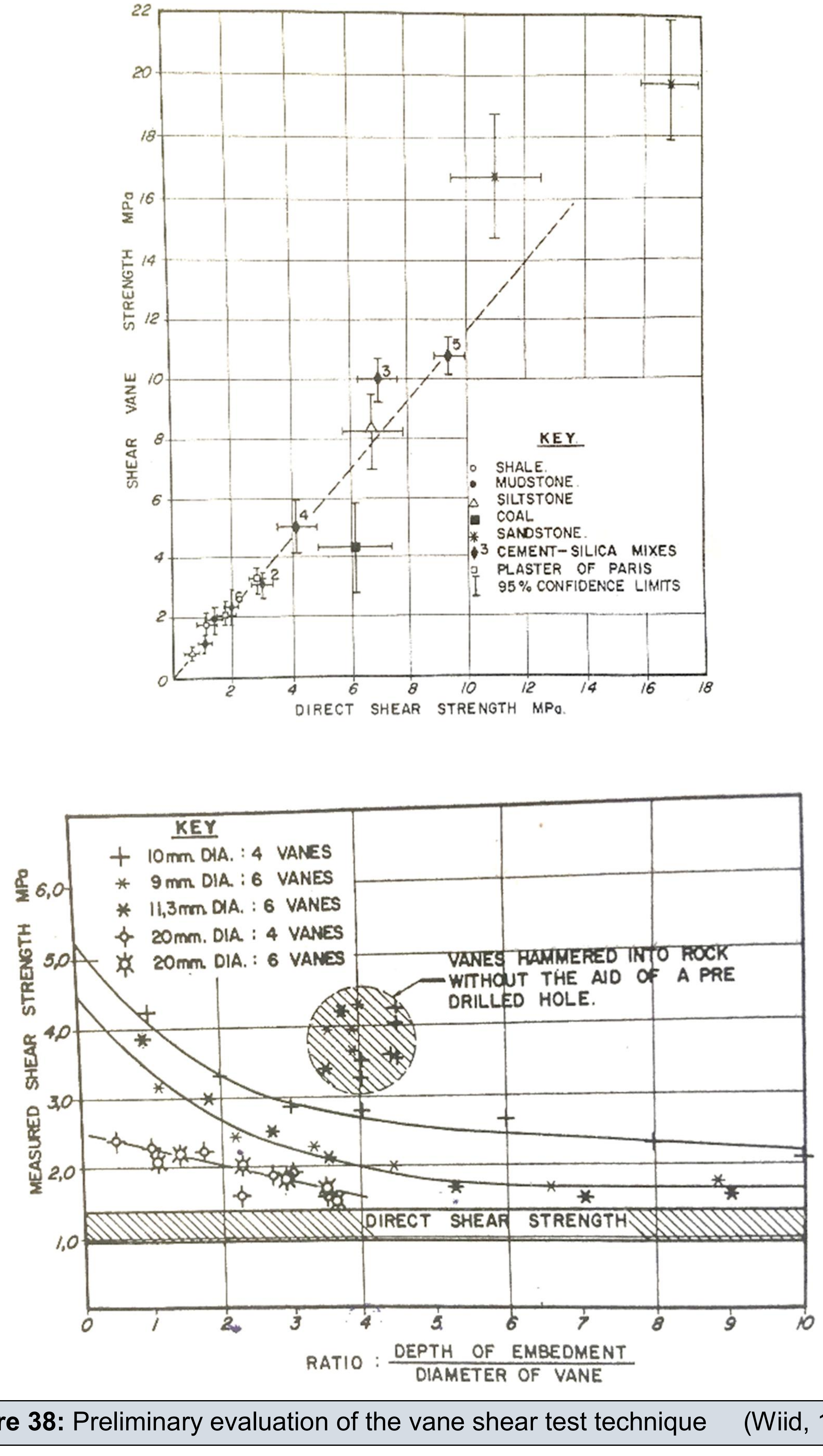


Figure 38: Preliminary evaluation of the vane shear test technique (Wiid, 1981)

Although this technique has not become available in commercial practice, it has great potential to provide reasonable shear strength estimates, and the portable equipment makes it possible to measure the shear strength of rock outcrops in the field, providing strength parameters within the early preliminary design stage.

Additionally, considering the limitations of accurate compressional strength estimations for soft rock, especially for mudstone where the point load test is considered unsuitable, this alternative may be quite useful.

A limited testing program was thus carried out to confirm the validity of the vane shear test technique.

For these tests, boulder size sample blocks were obtained from an opened quarry, which is used for surfacing aggregate on National Route 1, between Kroonstad and Ventersburg. Mudstone, sandstone and moderately weathered dolerite were sampled from the overburden. However, during the experimental work, it was found that the sandstone was too hard for the vane and the dolerite was not uniformly weathered. Only the mudstone was thus tested, and the results are presented in the following sections.

3.2 Sample description

The mudstone source comprises slightly to un-weathered, khaki, very fine-grained, clay-rich rock of the Beaufort Group, Karoo Supergroup. According to Brink (1983), this source is characterised by a tendency to break down rapidly after exposure to water or atmospheric change and excessive swelling. Additionally, the available mudstone blocks contained several microfractures which makes the cutting of samples for specialised laboratory testing very difficult.

3.3 Assumptions and Limitations

Due to limited resources, a quick and inexpensive method of validation was needed. With limited available testing techniques that measure the shear strength, a comparison between the measured vane shear strength results and the shear strength estimates, derived from compressional strength measurements, is considered. The correlation given in Equation 5 in Section 2.1.1 is used for the shear strength estimates from UCS values within this study.

Ideally the UCS values should have been measured with UCS tests. This would have required coring of the samples. Since the mudstone is known to slake in the presence of water, dry cutting methods would be required. Additionally, with the presence of micro cracks in the sample the required core length could not be guaranteed.

The Schmidt hammer was thus selected to estimate the compressional strength of the rock samples. The equivalent UCS value was estimated with the Figure given in the Appendix.

The vane shear equipment and procedure were fashioned to roughly mimic the experimental setup of Wiid (1981). However, although a minimum embedment depth of four times the vane diameter is recommended, the mudstone generally started to crack when the embedment depth was 2-3 times the vane diameter. When testing thinner sections, the strength estimates are expected to be generally overestimated.

3.4 Test setup and experimental procedure

The following equipment was selected for this study:

- The shear vane;
- Socket;
- Torque wrench, modified with attached wrench handle of similar weight in order to form a stable 2 handle torque wrench;
- Drill with 9mm diameter concrete drill bit;
- Nylon hammer;
- Schmidt Hammer;
- Table; and
- Straps.

The samples were strapped down on the table to prevent any movement. Several Schmidt Hammer strength readings were then taken around the area where the vane test would be carried out.

A hole was drilled and the shear vane embedded by hammering it in place with the nylon hammer. The torque was read from the analogue gauge and the strength estimated with Equation 8.

The tools used during the experimental work and some photos taken during the test procedure is given in Figure 39 and Figure 40 respectively.

		
Torque wrench	*Shear Vane*	*Schmidt Hammer*

Figure 39: Test equipment

		
Embedded vane	Failure along crack	Failed rock material

Figure 40: Vane test procedure

3.5 Test results

The findings of the experimental work are given in Table 4.

Recorded values where the rock failed along cracks, which formed during embedment, were excluded from this study.

Table 4: Experimental Test Results

Sample Block	*Schmidt Hammer*				*Vane Shear*	
	Readings	*Ave measurement*	*Equivalent UCS (MPa)*	*Equivalent shear strength (MPa)*	*Max torque (MPa)*	*Equivalent shear strength (MPa)*
1	31	33	58	9	20	10
	33					
	35					
	41	41	85	14	25	13
	41					
	41					
	44	42	85	14	30	16
	41					
	41					
2	41	38	80	13	25	13
	33					
	41					
3	43	45	110	18	30	16
	49					
	44					
	44	46	110	18	30	16
	44					
	49					

The correlation between the equivalent shear strength values from the respective test techniques are included in Figure 41.

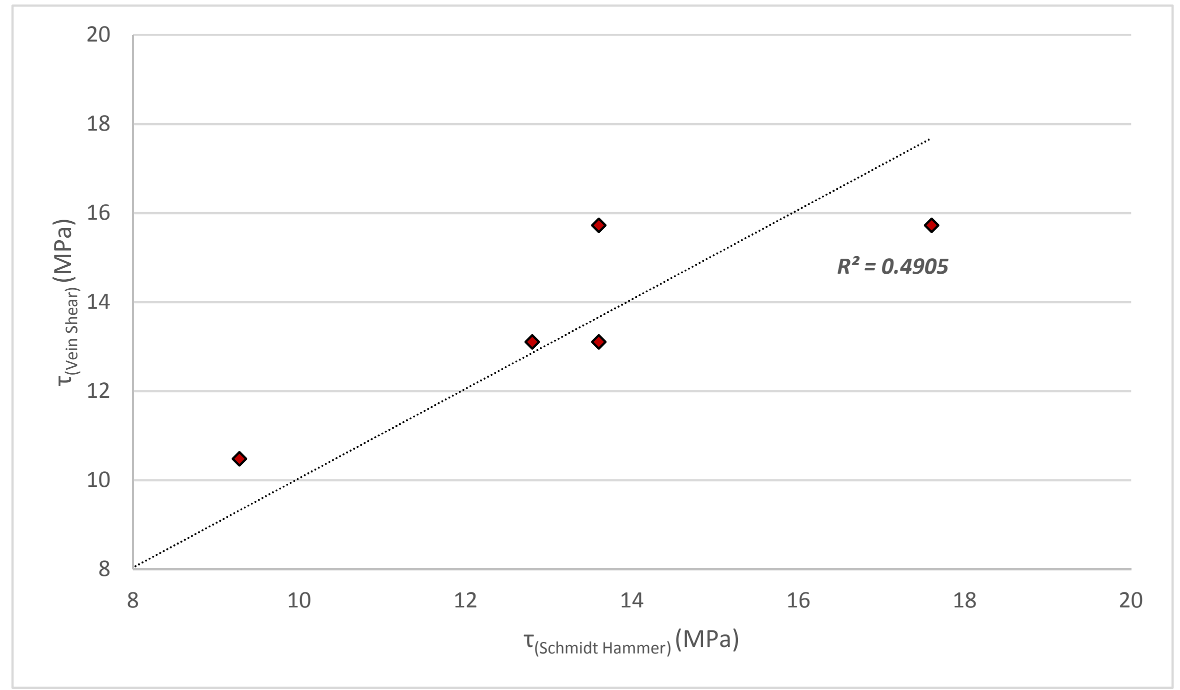


Figure 41: Experimental test results

3.6 Conclusion

Although the experimental procedure did not comprise a controlled environment and the validation was done using a technique with significant inherent variability, a reasonable correlation was still obtained.

The vane shear test is thus considered an inexpensive alternative technique to provide reasonable rock strength estimates quickly. Further work is, however, required to standardise this technique.

CHAPTER 4 - ANALYSIS

The rock (mass) response to a given design is estimated through models that are based on strength and stiffness estimates, obtained through the methods described within the previous chapters. These models adopt equivalent strength and stiffness characteristics for each portion of the rock (mass) that is representative of a specified stress environment.

Modelling practices for rock masses generally consider shear strength criteria. However, unexpected failures in major excavations (like deep pits) indicate the importance of damage mechanics and the presence of tensile strains in the rock (mass) that result in progressive failure at lower stresses than the conventionally predicted rock (mass) strength.

The general behaviour of the rock mass, based on measurable parameters is thus explored further below. This assessment is based on documented rock characteristics and relationships between measurable parameters from actual laboratory test results.

Relevant findings in the literature review, include:

- A consideration of rock strength that describes the rock (mass) response cannot be expressed by the rock hardness alone.
- Generally, hard rock is associated with brittle failure with associated crack initiation and propagation. However, rock material that is "soft" may also fail in a brittle manner under low confinement.
- Bieniawski (1967) estimated crack initiation at about 35% of the UCS value.
- Atkinson, et al. (1986) proposed a toughness index, which is an indirect measure of the rock fracture toughness and is estimated by a correlation between rock strength and stiffness. A toughness index of 27 is considered the limit of the rock to be cuttable (Tiryaki, 2006).
- Rock stiffness decreases with increasing strain.
- Additionally, Mair et. al. (1993), recognised that the resultant strain is not only dependant on the magnitude of the stresses, but also to the stress distribution.

- Generally soft rock is characterised by ductile deformation with predominantly shear failure. However, “hard” rock may also behave in this manner under high confining pressures.
- The complete shear strength envelopes of rock can be described based on the UCS test results, by considering:
  - $\sigma_1 = 3\sigma_3$;
  - The unconfined strength (σ_c) ≈ σ_3 for all rock types.
- Hoek (1999) found that rock behaviour is consistent with a soft rock when the UCS value is less than approximately one third of the in-situ stress acting on the rock.

Test results, including UCS, E_{50} and Poisson’s ratio measurements were taken (and the toughness index calculated) from the Wild Coast Project. These results are considered to represent the rock characteristics at 0.1% strain. Equivalent E_{50} values were then estimated for 0% and 1.0% strain based on Figure 19 in Section 2.1.4. Additionally, equivalent toughness index values were estimated for the adjusted E_{50} values.

Based on the literature review, together with an analysis of actual test results, the rock mass strength and associated expected behaviour are described in the following sections.

4.1 Rock (mass) strength under low confining pressures

A correlation between the strength of rock samples and their expected behaviour, due to the brittleness of the material, as well as the relationship between rock brittleness and the Poison’s ratio are explored in Table 5 and Table 6 respectively.

Rock with a toughness index higher than 27, comprises a material with a yield stress that is lower than its fracture toughness. These rock masses are considered to be hard, with failure predominantly controlled by the discontinuity network. On the other hand, rocks with a toughness index below 27 are considered soft, with failure both through the intact rock and along the discontinuity network.

Results in Figure 42 confirm the general understanding that soft rock is associated with low UCS values and hard rock with high UCS values.

However, rocks with similar UCS values have been tested as both hard and soft, even when the same rock type is considered. A possible explanation for this anomaly may be the influence of the Poisson's ratio on the expected rock behaviour (Figure 43). The study revealed an indirect relationship between Poisson's ratio and rock brittleness. Thus, rocks with a higher Poisson's Ratio will be characterised by material with a lower fracture toughness and thus a soft rock (mass).

Considering the reduced rock stiffness at higher strain levels (where the rock (mass) is subjected to elevated pressures), a soft rock (mass) may behave in a brittle manner. Here, failures are associated with fracture initiation, fracture propagation and progressive failure at stresses well below the strength estimations from the stress based failure criteria.

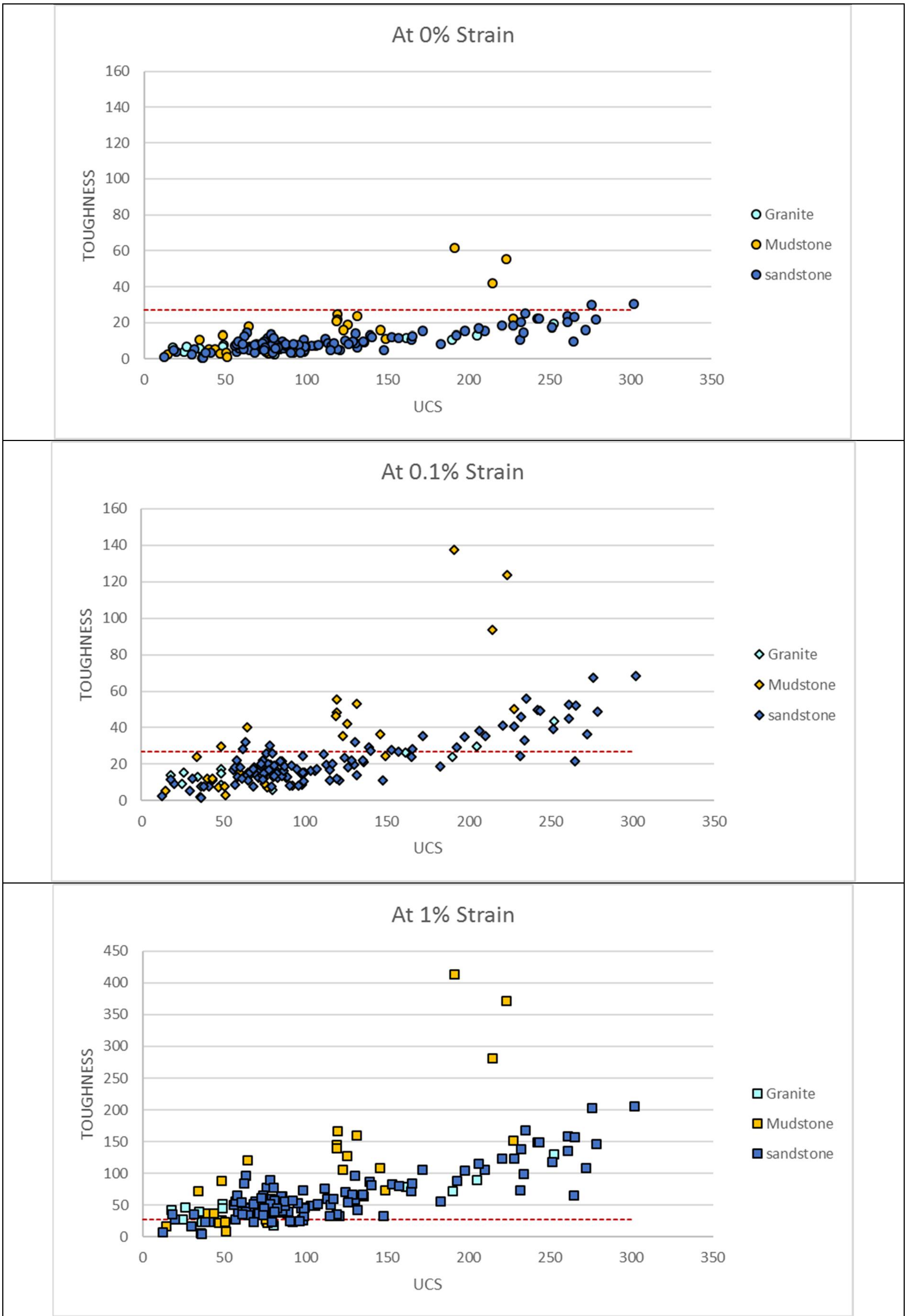


Figure 42: Rock toughness Index at various strain levels with varying UCS values

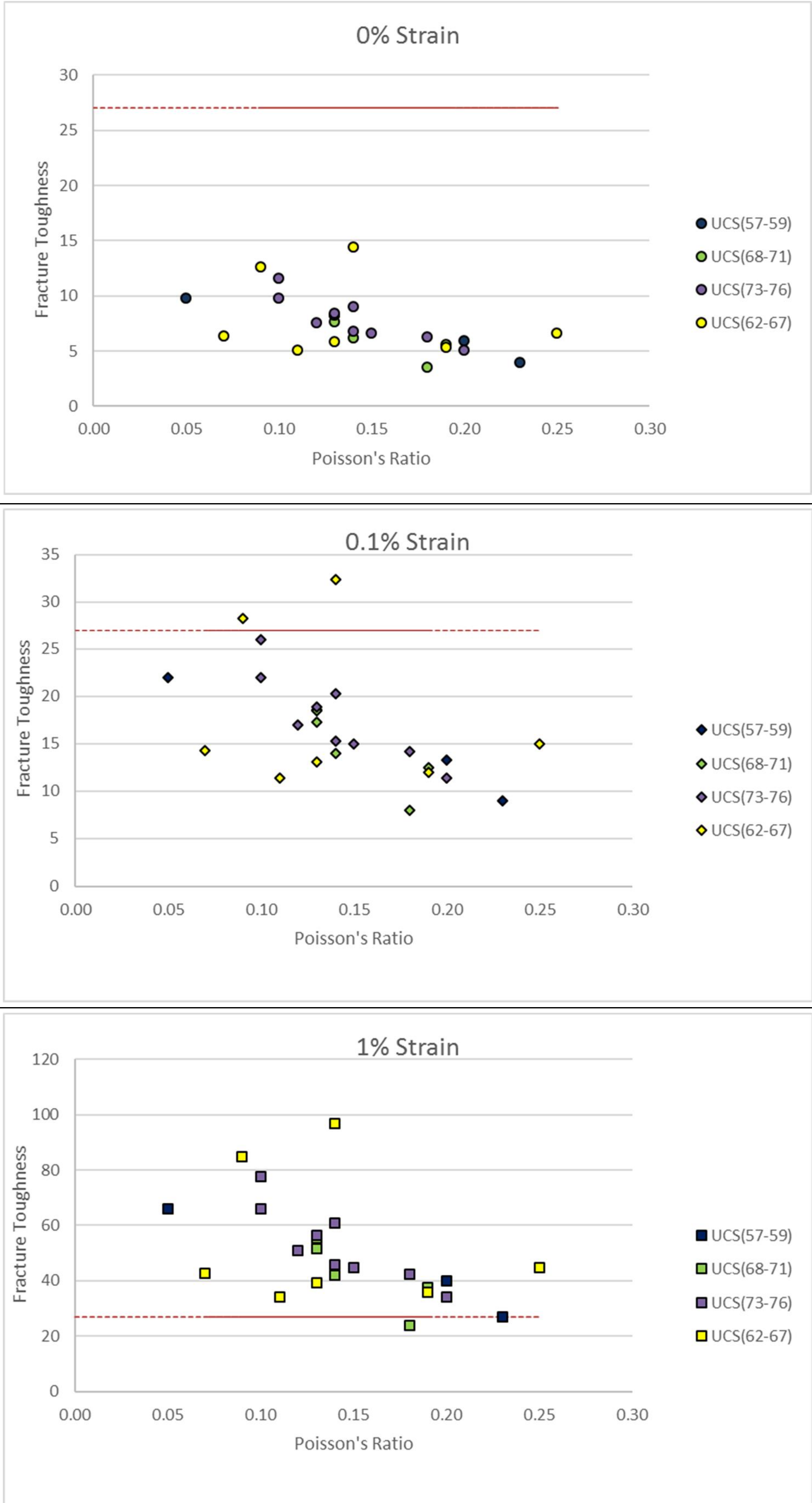


Figure 43: Rock toughness Index at various strain levels with varying Poisson's ratio

Based on the findings presented in Table 5, the general behaviour of mudstone and sandstone with increasing UCS values is proposed in Table 5.

<table>
<tr><th colspan="8">Table 5: Conceptional model for rock behaviour under low confinement</th></tr>
<tr><th colspan="2">Rock Hardness</th><th colspan="3">Mudstone</th><th colspan="3">Sandstone</th></tr>
<tr><td rowspan="5">Toughness Index</td><td></td><td>0% Strain</td><td>0.1% Strain</td><td>1% Strain</td><td>0% Strain</td><td>0.1% Strain</td><td>1% Strain</td></tr>
<tr><td>20 MPa</td><td><27</td><td><27</td><td><27</td><td><27</td><td><27</td><td><27</td></tr>
<tr><td>100 MPa</td><td><27</td><td>≤27</td><td>>27</td><td><27</td><td><27</td><td>>27</td></tr>
<tr><td>150 MPa</td><td><27</td><td>>27</td><td>>27</td><td><27</td><td>≤27</td><td>>27</td></tr>
<tr><td>300 MPa</td><td>>27</td><td>>27</td><td>>27</td><td>>27</td><td>>27</td><td>>27</td></tr>
<tr><td rowspan="5">Rock Mass Behaviour</td><td></td><td>0% Strain</td><td>0.1% Strain</td><td>1% Strain</td><td>0% Strain</td><td>0.1% Strain</td><td>1% Strain</td></tr>
<tr><td>20 MPa</td><td rowspan="3">1</td><td rowspan="2">3</td><td>5</td><td rowspan="3">1</td><td rowspan="3">3</td><td>5</td></tr>
<tr><td>100 MPa</td><td rowspan="3">6</td><td rowspan="3">6</td></tr>
<tr><td>150 MPa</td><td rowspan="2">4</td></tr>
<tr><td>300 MPa</td><td>2</td><td>2</td><td>4</td></tr>
</table>

LEGEND

<27; Soft rock

>27; Hard rock

1; "*Low stress environment*"- Failure comprise combination of failure through rock material and along discontinuity planes.

2; "*Low stress environment*"- Failure controlled by discontinuity network.

3; "*High stress environment*"- Failure predominantly through shearing (in accordance with Hoek-Brown Criterion).

4; "*High stress environment*"- Failure associated with fracture propagation and progressive failure at stresses well below the strength estimations from Hoek-Brown.

5; Excessive deformation, with squeezing of the tunnel.

6; Extension crack development, with slabbing of tunnel face.

4.2 *Rock (mass) strength under high confining pressures*

Rock that is considered to be hard under low confinement, is known to behave like a soft rock (mass) under high confining pressures. The thickness of the brittle zone depends on the rock properties.

Considering that the yield strength of the material (σ_1) = 3σ_3 and σ_c (which is the UCS value) ≈ σ_3, for all rock types, the critical pressure where a hard rock will start to undergo ductile deformation is roughly three times the UCS value. This concurs with Hoek's (1999) findings that a rock is soft when the UCS value is less than approximately one third of the in-situ stress acting on the rock.

Studies by Boccaccini (1997) on glass indicated that, as the rock brittleness increases, the fractured fragment sizes also increase. It is thus reasonable to assume that the distance between extension fractures will become less as the brittleness of the material decreases. Additionally, zones of "disking" of core are not uncommon and are explained by zones of high stresses. An additional explanation might be "disking" over the contact zone where the critical confining pressure is reached.

The proposed rock behaviour with increasing confining pressure is shown graphically in Figure 44 below.

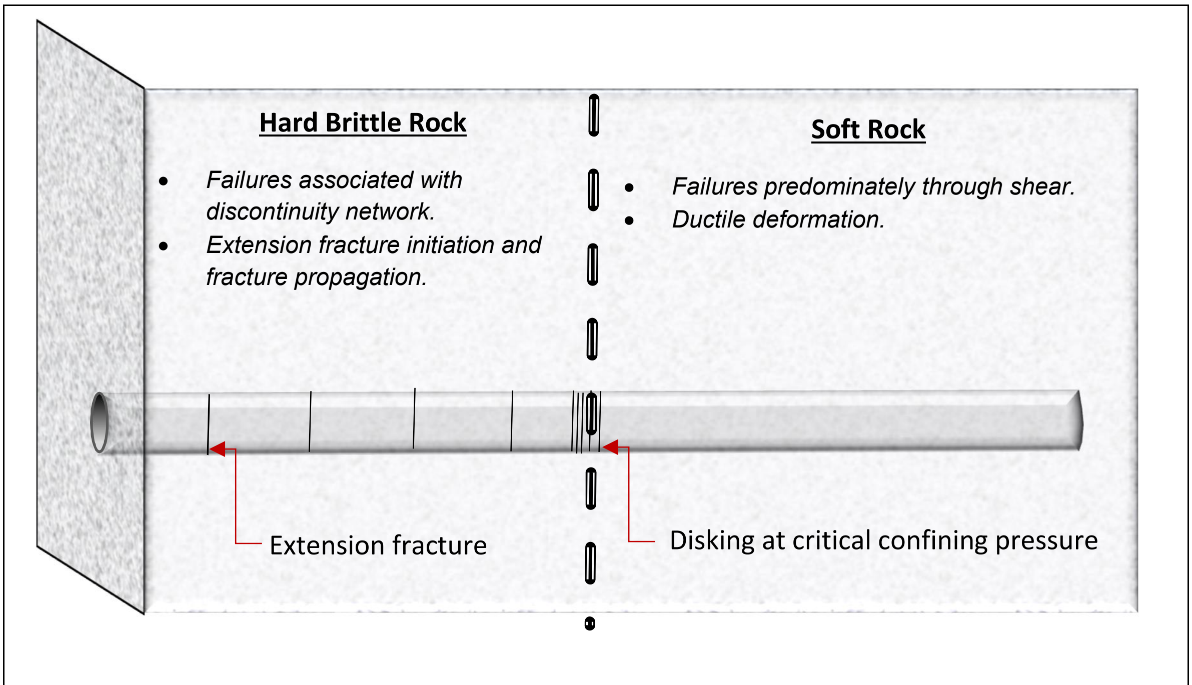


Figure 44: Conceptional model for expected rock behaviour with increasing confining pressures

CHAPTER 5 - CONCLUSION & RECOMMENDATIONS

A weak rock mass embodies a collection of material with diverse characteristics. These generally comprise:

1. *Soft Rock*

   There is no known standard definition and the description of soft rock varies between disciplines. Engineers describe the range of soft rock material based on arguments like the strength of concrete (25 MPa) and the refusal of the SPT spoon (~0.5MPa). Geologists describe soft rock based on the ability of the material to be scraped or peeled with a knife (up to 10 MPa) (Brink and Bruin, 2002).

2. *Bimrocks*

   Rocks associated with certain geological settings or weathering may result in variable masses with hard fragments in a soft matrix. Material that is characterised by UCS_{block}/UCS_{matrix} >1.5 is considered a bimrock (Medley and Zekkos, 2011).

3. *Fractured rock mass*

   A rock mass comprises an intact rock material that is weakened by a discontinuity network. Hard rock is generally considered weak due to fracturing when the RQD is less than 25% (Santi, 1997).

Rock hardness alone is insufficient to describe weak rock masses. Consideration should also be given to:

1. *The Stress Environment*

   <u>Low confinement</u>

   Rock mass failure is mainly controlled by its discontinuities. Within a highly stressed environment, soft rock behaves in a brittle manner, with possible crack initiation and propagation. Rock in this environment is known to deform under a constant load over time (commonly known as rock creep). Creep is generally encountered at a stress level of 40 – 60% of the rock strength.

High confinement

The rock strength and elastic properties (i.e. brittle behaviour) decrease as the pressure and temperature increase (He, 2014).

Even hard rock is known to undergo ductile deformation under high confining pressures. Hoek (1999) considers rock as soft when the rock compressive strength is less than approximately one third of the in-situ stress acting on the rock. The influence from the discontinuity network diminishes with increasing confinement. A rock mass under high confinement is thus expected to have similar characteristics to the in-situ rock material.

2. *Rock Durability*

Some rock types provide additional challenges associated with rapid weathering when exposed to the atmosphere. This is predominantly influenced by the rock composition. The following is generally an indication of soft rock (Santi, 2006):

- Slake durability <90%;
- Mudstones with less than 56% quartz, with additional deleterious material including smectite (Paige-Green (2005) suggests a limit of 10%) and sodium.

3. *Rock Stiffness*

The deformation of the rock under specified stress conditions prior to failure is described by the rock stiffness. General trends for rock stiffness include:

- Stiffness decreases with increasing strain.
- There is a direct relationship between rock strength and stiffness that seems to be independent of the physical rock properties.
- For rock masses, the deformation modulus is also affected by the mode of failure, with gradients on the log–log plot ranging between 1.4 for sliding and 1.8 for shearing (Ramamurthy, 2004). However, the direct correlation is only applicable for rock masses where the joint orientations are between 0° and the critical angle with respect to σ_1. In rock masses with joint orientations exceeding the critical angle, there appears to be an inverse relationship between rock strength and stiffness.

4. *Groundwater*

   Rock strength is known to reduce with increased water content and this depends on the rock type:

   - The effect of water content appears to be more significant in sedimentary deposits compared to crystalline rock (Wong et. al., 2016).
   - S*andstone* - A sudden decrease in the strength at a moisture content between 0% and 1% (Agustawijaya, 2007); and
   - *Mudstone* - Gradual degradation up to saturation (Jumikis, 1966);

   The water flowing through the discontinuity network does not significantly impact on the rock mass strength, but rather acts as a destabilising agent.

5. *Block proportion for Bimrocks*

   In bimrocks, the shear strength of the mass is greater than the shear strength of the matrix. This is believed to be related to a tortuous failure path around the rock fragments.

   The shear strength of bimrocks are thus related to the block proportion as follows:

   - When the proportion of hard blocks is less than 25 %, the strength of the rock mass is generally similar to the strength of the rock matrix (Santi, 2006);
   - Work by Santos, et al. (2002) shows an apparent increase of the internal friction angle at a rate of about 2.5° for each additional 10% of blocks;
   - The increase in the overall friction strength can be up to between 15° and 20° above the matrix friction strength (Medley and Sanz Rehermann, 2004) and there appears to be no additional increase in friction angles when the block proportion exceeds 70 % (Medley and Zekkos, 2011).
   - Work by Lindquist and Goodman (1994) indicated a decreased cohesive strength in relation to the increase in internal friction angle (Haneberg, 2004).

   However, a preliminary study by Sönmez, et al. (2004) revealed an apparent relationship between the overall strength of the bimrock and the UCS of the blocks. Thus, the general assumption that overall bimrock strength is related to a tortuous failure path around blocks, may be an oversimplification.

Available tools to estimate the rock characteristics include:

1. *Laboratory tests*

Laboratory techniques are limited to small samples and generally provide estimates for the *intact rock material*, which is known to overestimate the material strength in the field by up to 30%. The exception being bimrocks, which are scale independent. Available laboratory techniques are summarised in the table below:

Table 6: Laboratory Test Techniques			
	Rock Strength	***Rock Stiffness***	***Rock Durability***
Direct Methods	**UCS** – Index test; • Most commonly used test in commercial practices. • Require core of sufficient length. • Results from tests carried out under standardised loading rates, are generally grossly overestimated **Brazilian Test** Measures tensile strength **Triaxial test** – Specialised test is time consuming and relatively expensive, but give reasonable strength and stiffness estimates	**UCM** – Index test E is measured at 50% UCS **Triaxial test** - Specialised test	**Wetting and Drying** Nickmann, et al. (2006) created a useful classification system Refer to Section 2.1.2 **Slake durability test**
Indirect Methods	**Point load** Despite natural scatter associated with correlations with this technique and soft rock's tendency to deform significantly before failure, this technique is used extensively in industry. Recommended correlations: • $UCS_{(Crystalline\ rock)} = 23 \times Is_{(50)}$, • $UCS_{(Sandstone)} = 14 \times Is_{(50)}$, • Mudstone / shale – Should not be used. **Punch tests** Has ability to evaluate rock material anisotropy, but it is not commercially available in South Africa. Recommended correlation; $UCS = \varepsilon BPI_c$ (ε = 5.1-5.5, but 5.25 recommended for mudstone / shale) **Vane shear** Currently not a standard test technique for rock, but is considered an inexpensive alternative to determine reasonable rock strength estimates quickly	-	**Mineralogical analysis** Indications of material prone to slaking: • Mudstones with less than 56% quartz; • Rock with more than 10% Smectite; • The presence of deleterious elements includes iron sulphides with chlorite and sodium.

When the rock material is too soft to be tested with rock testing techniques and too hard to cut and test as a soil, it is useful to know that the internal friction angle of in-situ rock and that of the remoulded sample tend to be similar.

Rock brittleness may be estimated through a correlation between UCS (or estimated UCS values with indirect methods) and the Brazillian test results.

Rock fracture toughness index is estimated through a correlation between UCS and E. A toughness index of 27 is considered the limit of the rock to be cuttable (Tiryaki, 2006).

2. *Field evaluation Techniques*

With these techniques, the boundary conditions are often not known, which creates a degree of uncertainty in the interpretation of the results.

Available field techniques are summarised in the table below:

Table 7: Field Evaluation Techniques

	Rock Strength Only	***Rock Stiffness Only***	***Rock Strength & Stiffness***
Direct Methods	-	**Plate Load Test** This is the most documented field technique, that has been used extensively for large projects throughout the world. **Flat Jack Test**	**Pressuremeter Test** This test is carried out in boreholes. Limitations: • Radial cracking affects the stiffness estimate, and • The test results are sensitive to the presence of soft seams in the area tested.
Indirect Methods	**Rock Mass Classification Systems** • The Q system considers the degree of fracturing, the fracture network characteristics and the stress environment; • RMR considers intact rock strength, degree of fracturing and the fracture network characteristics; • GSI (which is based on RMR) considers degree of fracturing, the fracture network characteristics and disturbance due to blasting. • When soft rock masses are considered, only the RMR has the capability to adjust the rating accordingly.	**Seismic surveys** The shear wave velocity is correlated with the rock stiffness, according to: $G_0 = \rho V_s^2$ **CSW Test**	**Empirical Formulae** Available empirical correlations to estimate rock mass strength and stiffness are given in Table 1 The criterion that best suit the site conditions should be selected. Only use this in the absence of in-situ data for preliminary designs.

Modelling practices for rock masses generally consider shear strength criteria. The complete shear strength envelopes of rock can be described based on the UCS test results, by considering:

- $\sigma_1 = 3\sigma_{3;}$
- The unconfined strength (σ_c) ≈ σ_3 for all rock types.

However, unexpected failures in large excavations indicate the importance of damage mechanics and the presence of tensile strains in the rock (mass) that result in progressive failure at lower stresses than the predicted rock (mass) strength.

Within this study, rock strength is defined by its response as:

- Hard Brittle Rock
  - Failures predominantly through discontinuity network within a low stressed environment;
  - Extension fracture initiation and fracture propagation in a highly stressed environment.

- Soft Rock
  - Failures through both rock material and discontinuity network within a low stressed environment;
  - Failures predominantly through shear in a highly stressed environment;
  - Excessive ductile deformation within high confining pressures.

Through the consideration of trends between rock strength (UCS), fracture toughness, Poisson's ratio and rock stiffness at increasing strain, the following was observed:

- Soft rock is associated with low UCS values and hard rock with high UCS values.
- However, exceptions may be found where a rock with a high UCS value behaves as a soft rock. This is considered to be due to an indirect correlation between Poisson's Ratio and rock brittleness (i.e. Rock with

a high Poisson’s Ratio will be characterised by material with a low fracture toughness and thus a soft rock response).

- Additionally, a hard rock under low confining conditions can behave as a soft rock under high confining pressure. The critical pressure from which the rock is considered to be soft is considered to be three times the UCS value.

Available testing techniques to provide index strength and strain estimates comprise essentially robust methods, where an accuracy within 10 – 20% is considered acceptable. Similar results obtained from several tests on the same material generally give a degree of confidence in the results.

Additionally, technological development within this modern era has led to sophisticated modelling software packages, which are powerful tools for design and generally add to the degree of confidence in the predictions.

With these advances, there was a shift in focus from experience and engineering judgement to accurate behavioural analysis, through analytical techniques. Although available modelling tools are powerful and have great potential to make behavioural predictions, they should not be used blindly. Engineers should be cautious not to misinterpret precision for accuracy.

This report considered a Modified Vane Shear Test, proposed by Wiid (1981) for the measurement of shear strength of soft rock. A limited study showed this test could be considered as an inexpensive alternative technique to provide reasonable rock strength estimates quickly. It is recommended that further research work should be carried out using the shear vane, to be able to standardise this technique.

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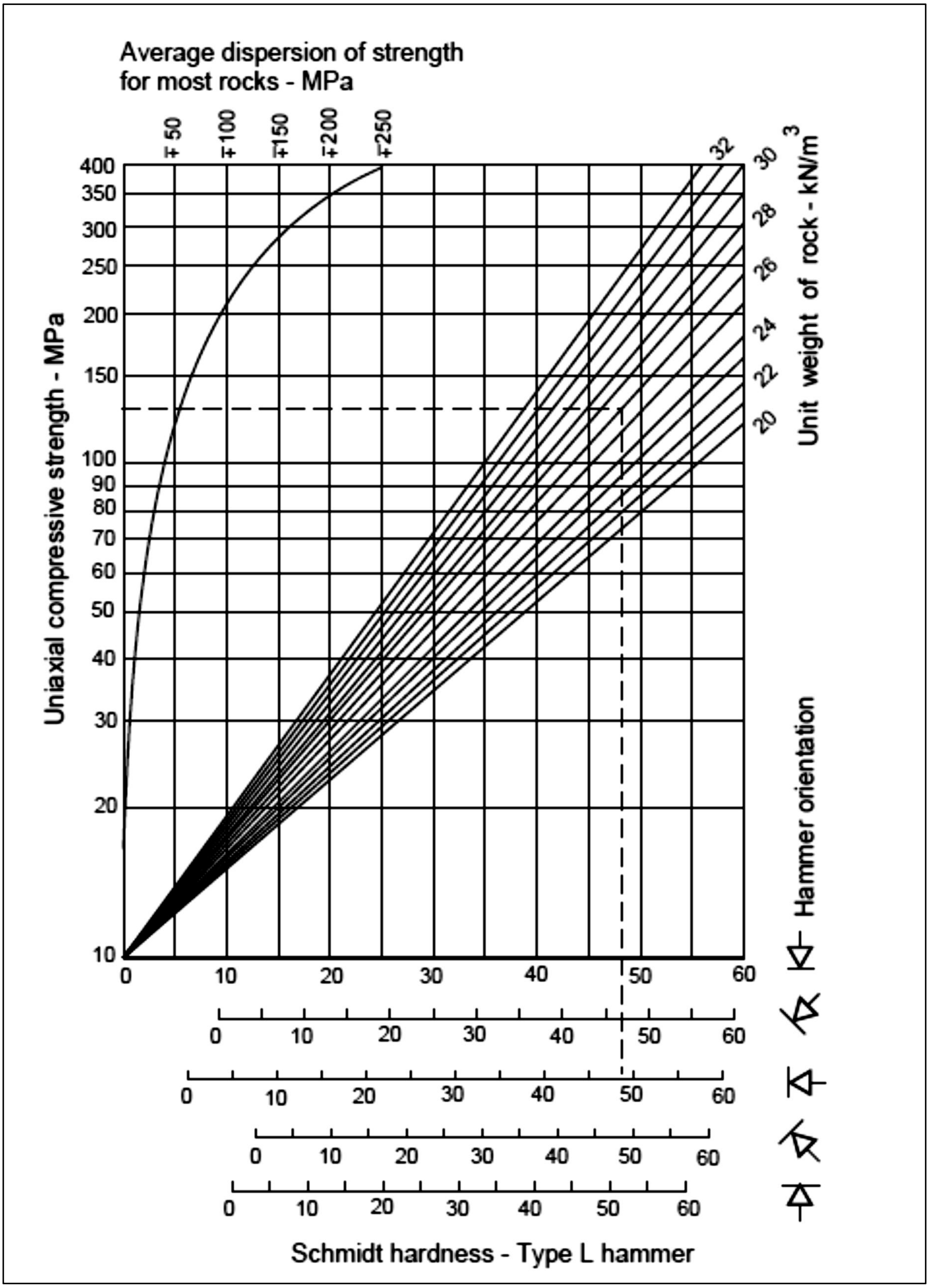
Appendix A



(Hoek, 2007)