

EVALUATION OF STIFF TESTING OF ROCK

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DECLARATION

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

.....

(Signature of Candidate)

..... day of year

ABSTRACT

When designing structures in rock, accurate and relevant information on the mechanical properties of the materials encountered is essential. While laboratory testing of rock materials provides some of the required data, it is important that the reliability and limitations of the information gained are fully understood. Tests on specimens with different height-to-diameter ratios from two materials of Class I and Class II type were evaluated to obtain a better understanding of the deformation behaviour of rocks in the post-peak region of the complete stress-strain curves. It was shown that the geometry of the test specimens influence the stress-strain behaviour in the post-peak region. The numerical values for the slope of post-peak graphs are thus not material constants per se, but are geometry dependent. Highly localized failure planes were observed for all the tested specimens with Class II type behaviour. Explanations for the observed phenomena have been proposed.

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LIST OF SYMBOLS

Brittleness index	k
Diameter	Φ
Displacement	D
Force	F
Joule	J
Modulus of elasticity	E
Post-peak modulus	M
Poisson's Ratio	ν
Stiffness of an elastic body	K
Strain	ε
Strain (axial)	ε_a
Strain (lateral or transverse)	ε_l
Strain (millistrain = 0.001 m/m)	$m\varepsilon$
Stress	σ
Transverse deformation modulus	T

CHAPTER 1

INTRODUCTION

1.1 Background

Engineering structures in and on rock have become ever more important for the improvement of the quality of life, for creating wealth and even for the survival of humankind. Mines reaching ever greater depths, water storage dams and transport and water tunnels being constructed in ever escalating numbers and at the same time having larger excavated cross sections and in addition often having to be built under increasingly unfavourable conditions have required more and better information on the building material - rock. Fortunately, the development of hardware and software for computer modelling of the excavations has provided improved tools for rational designs. In turn, however, these powerful modelling techniques require relevant input data and thus, inter alia, accurate and reliable values for the mechanical properties of rocks.

Data on the properties of rock materials such as strength and deformation characteristics are obtained by laboratory tests. The tests should always be performed according to standard procedures. Examples of such test procedures are the recommendations by the International Society for Rock Mechanics (ISRM), the American Society for Testing and Materials (ASTM) and national

standards e.g. British Standards (BS) or Deutsche Industrie Normen (DIN). These recommendations are based on current best practice. In spite of these standards which are essential for comparison of data, there are still certain aspects of testing which require further attention.

It is well known that the response of rock materials to forces is one of the most basic and important properties when studying the behaviour of rocks. This response is usually presented by stress-strain curves from which the deformation characteristics are derived. While the pre-peak behaviour of the stress strain curves are used routinely, the behaviour in the post-peak region of the stress-strain curves is not yet fully understood. Although the rock has failed after the peak stress (strength) has been reached, failed rock is still able to withstand forces as may be observed in for example mining excavations. Therefore, information on this post-peak behaviour is a necessary requirement for modern computer modelling of excavations in rock. However to attain this information, tests for the determination of the post-peak properties require sophisticated and expensive equipment. Often these tests are not successful, particularly when strong, brittle rock materials are tested.

1.2 Aim and objectives

This dissertation deals with research into the deformation and failure behaviour of rock materials by means of stiff, servo-controlled testing equipment. The particular aim of the study was to test the premise that the behaviour of cylindrical rock test specimens in the post-peak region of the stress-strain curves is not only a characteristic material property but is influenced by the height-to-diameter ratio. An additional objective was to investigate the role of stiffness of the test equipment on the outcomes of tests. Personal knowledge gained through over forty years of involvement with laboratory testing of rock materials has been applied when critically commenting on testing procedures.

1.3 Layout of the dissertation

In Chapter 1 of this dissertation the background for this study, the aim and objectives and the layout of the dissertation are presented.

In Chapter 2, an extensive overview of the literature pertaining to the testing of rocks is summarized focusing on deformation properties and on test methods and equipment. In addition, the experimental variables affecting the outcomes of tests are listed and discussed. Comments on aspects of testing have been added where applicable.

In Chapter 3, the testing programme and the experimental procedures are presented which entailed the testing of two types of materials, a weak, model material with Class I behaviour and a medium-strong Anorthosite with Class II behaviour. Compression tests on cylindrical specimens with different height-to-diameter ratios were performed. The results of the tests are presented and the conclusions made are discussed.

In Chapter 4, the knowledge gained during the investigations is listed and discussed. Comments on the method of deformation measurements and on the phenomenon of ‘highly localized failure planes’ are made. Also the role of transverse deformations on post-peak behaviour is investigated.

In Chapter 5, the conclusions summarizing the results of the investigations are presented. An explanation for the observed behaviour, i.e. the effect of the elastic energy stored in the unfailed fragments of the test specimen which is released as the load bearing capacity of the specimen diminishes, is given. Recommendations based on the results and their interpretations are suggested.

CHAPTER 2

LITERATURE STUDY

In this section a summary of the relevant available information is presented. The pre-peak as well as the post-peak stress-strain relationships illustrating the deformation characteristics of rock materials are discussed. Testing equipment and methods of controlling the equipment during tests are described and factors affecting the test results are presented. Where appropriate, comments have been added.

The results of tests on a given rock material depend on a large number of experimental variables such as:

- The quality of test specimen preparation
- The diameter of the test specimen
- The ratio between the length and the diameter of the test specimen
- The rate of loading
- Moisture content, etc.

Standardized test methods had to be developed to ensure comparable test results regardless of where the tests have been performed. Some of the internationally best known standards for testing of rocks are the test methods suggested by the

International Society for Rock Mechanics (ISRM SM) and the standards of the American Society for Testing and Materials (ASTM). Examples of such standards are ISRM, (2007a); ISRM, (2007b); ASTM D2664 and ASTM D7012.

2.1 Stress-strain behaviour of rock in the pre-peak region

The most common approach for studying the behaviour of rock in the testing laboratory is by compression testing of cylindrical specimens specially prepared for the tests. Rocks respond to forces by deforming and eventually breaking. Properties derived from the stress-strain curves produced are the strength and the deformation characteristics. The strength being the maximum stress a test specimen can withstand which is equivalent to the peak of the stress-strain curves. The deformation characteristics are usually expressed as modulus of elasticity (E) and Poisson's ratio (ν). E is also referred to as Young's modulus or even simply as Modulus. If the axial stress-strain curve is linear-elastic, then only one constant value for E will describe the behaviour. If, however, the axial stress-strain curves are non-linear the values for E depend on the method of their determination. In addition, the values may depend on the stress level or the stress interval over which the evaluation is done. Both the ISRM SM (ISRM, 2007a) and the ASTM (ASTM D7012) suggest three methods. Figure 2.1 below illustrates the methods for calculating Tangent Modulus, Average Modulus and Secant Modulus respectively. For evaluating the Poisson's ratio, the same methods are suggested.

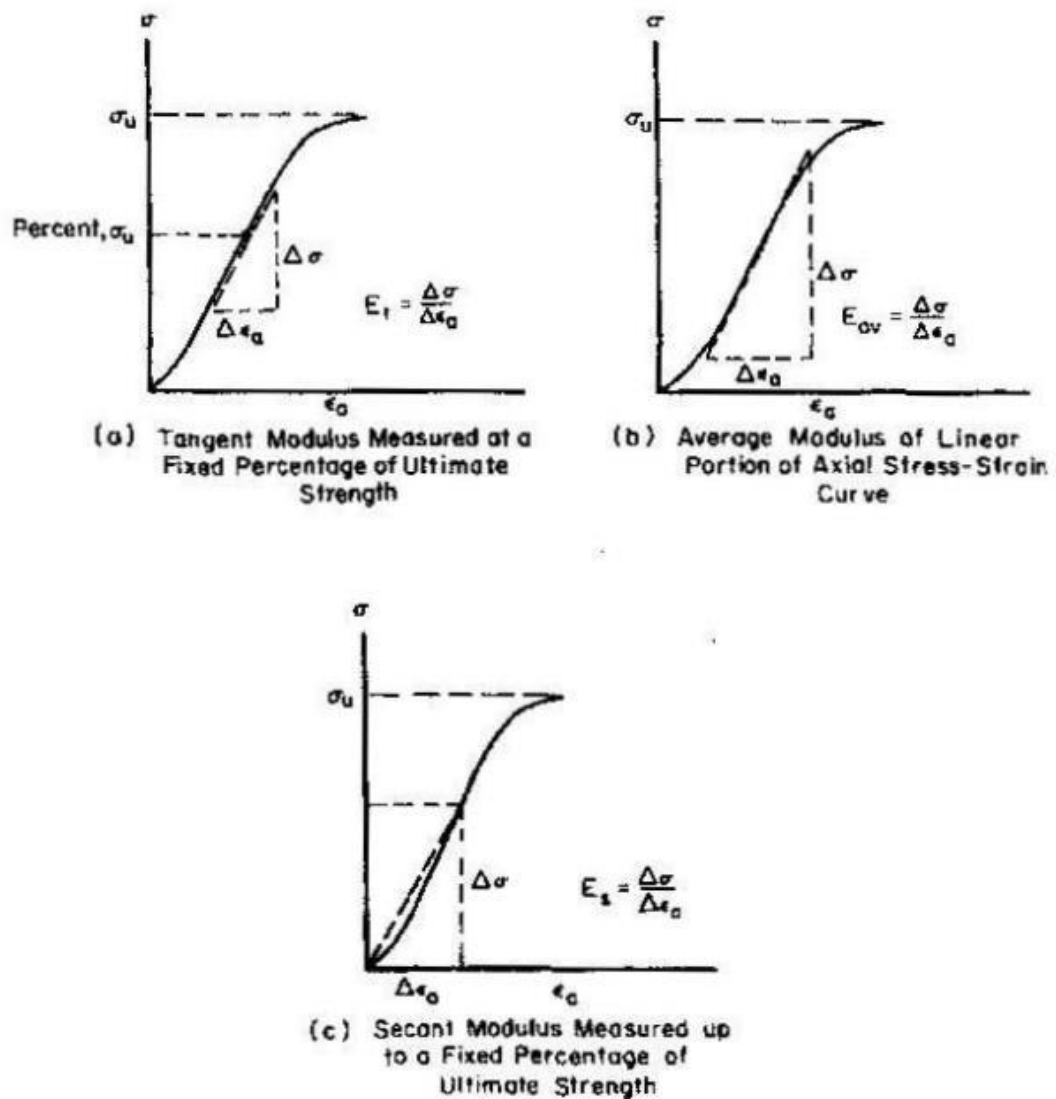


Figure 2.1 *Methods for calculating Young's modulus from axial stress-strain curves (after ISRM, 2007a)*

When reporting results derived from non-linear stress-strain curves the method of determination should therefore always be stated. In South Africa it is common practice to evaluate and report the Tangent modulus calculated at 50% of the ultimate strength. However, should the E modulus be required for, say the evaluation of in-situ stress measurements, a Secant modulus calculated from data spanning the appropriate stress interval may be more relevant.

The standard tests for the determination of the deformation behaviour of rocks in uniaxial compression recommend one uninterrupted loading cycle up to the stress level of rupture (ISRM, 2007a and ASTM D7012). Standard shapes and sizes for the test specimens and standard loading rates are recommended. The ASTM uses the term 'length' whereas the ISRM uses 'height' to indicate the dimension of the test specimens in the axial direction. In this dissertation the terms 'height' and 'height-to-diameter ratio' will be used.

The typical outcome of a uniaxial compression test as described by Bieniawski (1967c) is illustrated in Figure 2.2 and is discussed below.

A number of distinct phases (numbered 1 to 4) separated by limiting stress levels (numbered I to V) may be noted.

- During phase 1, between zero stress and stress level I, the axial curve is concave. This phenomenon has been ascribed to 'crack closure' during which phase the rock offers gradually increasing resistance to deformation. For very strong rocks this phase may be absent.
- During phase 2, between stress levels I and II, both axial and lateral and hence the volumetric strains are linear. The values for modulus E and Poisson's ratio ν are therefore constant.
- At stress level II, the level of fracture initiation, micro cracking will start.
- During phase 3, above stress level II, micro cracking will continue and gradually accelerate. This will cause an increase in lateral strain rate resulting in concave behaviour of the volumetric strain curve. At the end of this phase, at stress level III, the onset of unstable fracture propagation also termed the 'critical energy release' is reached.
- Stress level III also indicates the onset of volumetric dilation.
- During phase 4, above the level of critical energy release, both axial and lateral strain rates increase rapidly, eventually leading to strength failure.

The lateral strain rate increases far more rapidly than the axial rate, resulting in the volumetric curve changing direction from decreasing to increasing volume.

- Fracturing or crack formation and extension takes place during both phases 3 and 4. However, during phase 3 further crack formation will be halted when the applied stress is kept constant, while for stresses above the level of 'onset of unstable fracture propagation' (level III) crack formation will continue with time, even under constant stress. This will eventually lead to strength failure.
- Stress level IV indicates the maximum stress or strength of the specimen.
- Final rupture is indicated by stress level V. This is when the specimen loses all further resistance to load.

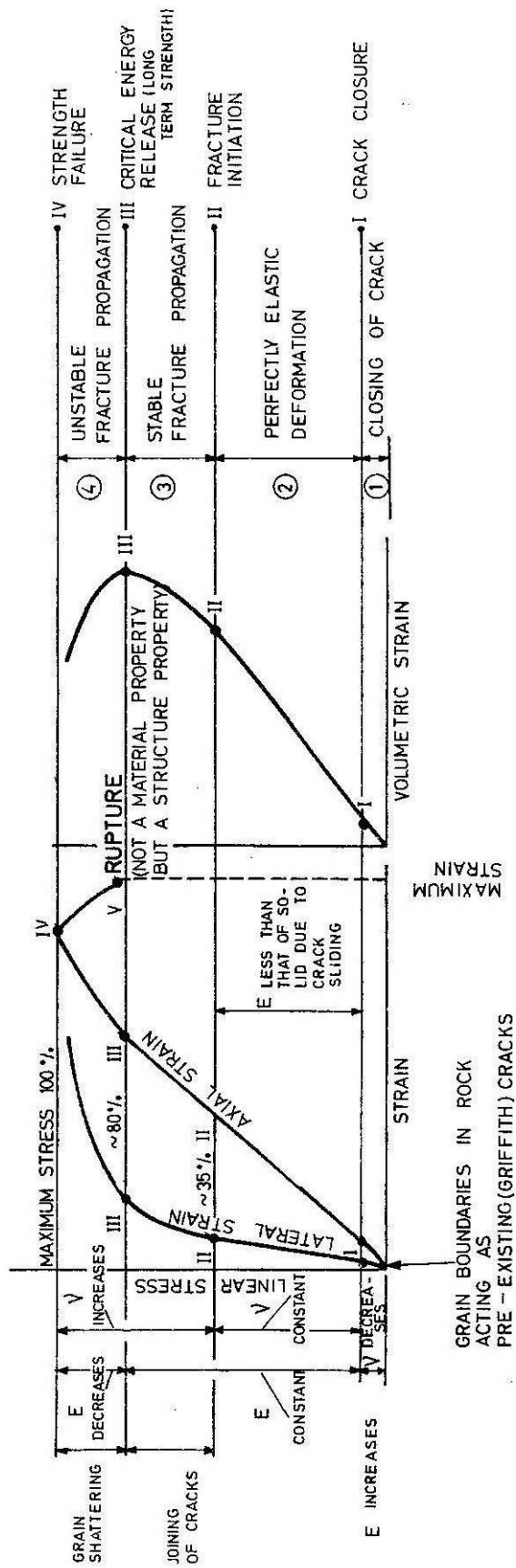


Figure 2.2 Stress-strain behaviour of rock (after Bienawski, 1967b)

Bieniawski (1967c) has successfully linked the stress at the ‘critical energy release’ (stress level III) for a Norite to the long term strength as shown in Figure 2.3 below.

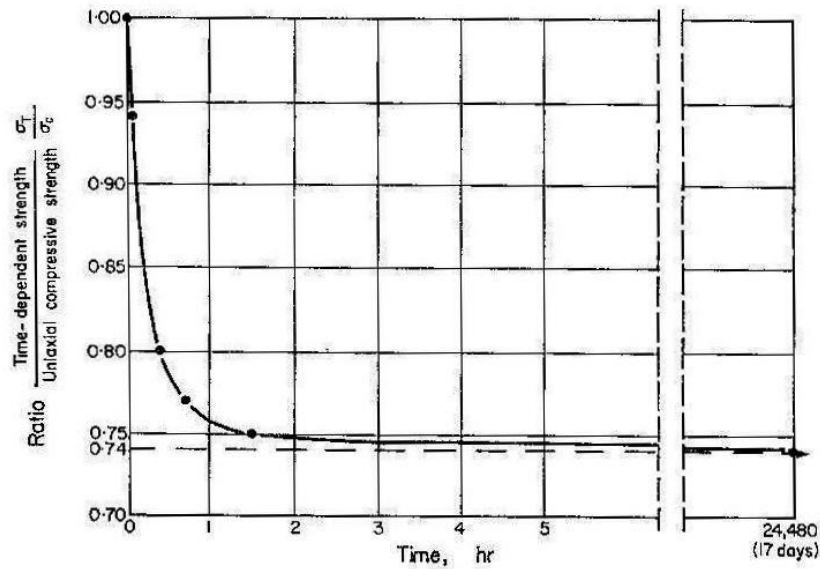


Figure 2.3 Time dependent strength of Norite (after Bieniawski, 1967c)

The points on the graph of figure 2.3 were obtained by loading a number of test specimens, each to the desired stress level. These stress levels were then kept constant and the time taken for the specimens to fail was recorded. It is important to note that the long-term stress level, above which the abovementioned Norite will eventually fail, is about 74 % of its short-term strength as determined by conventional laboratory tests.

Rocks have traditionally been regarded as brittle materials because maximum load bearing capacity and the stress at the point of rupture have been observed to be the same, or practically the same, and in addition, rupture generally is in a violent, explosive mode. However, with the advent of better testing equipment combined with the development of better testing procedures, it was observed that the stress at rupture (stress level V in figure 2.2) does not necessarily coincide with the stress at strength failure (stress level IV in figure 2.2) . One such development was the introduction of deformation or strain control instead of load or stress

control when applying load to the test specimens. This led to the realization that a uniaxial compression test specimen does not necessarily have zero load-bearing capacity after its strength failure. In the mid 1960's Cook (1965) concluded that the violent, brittle behaviour was due to excess energy stored in the elastic components of the testing machine. As a result of this insight stiff testing machines were developed.

Cook and Hojem (1966) at the South African Chamber of Mines Research Organization (COMRO) developed a stiff testing machine going as far as using mercury as hydraulic fluid.

Bieniawski et al (1969) at the South African Council for Scientific and Industrial Research (CSIR) used a technique by which stiff steel columns were loaded in parallel with the rock test specimen. The aim of these columns was to absorb the excess elastic energy stored in the testing machine when the rock specimen starts to lose its load bearing capacity. This set-up is illustrated in Figure 2.4.

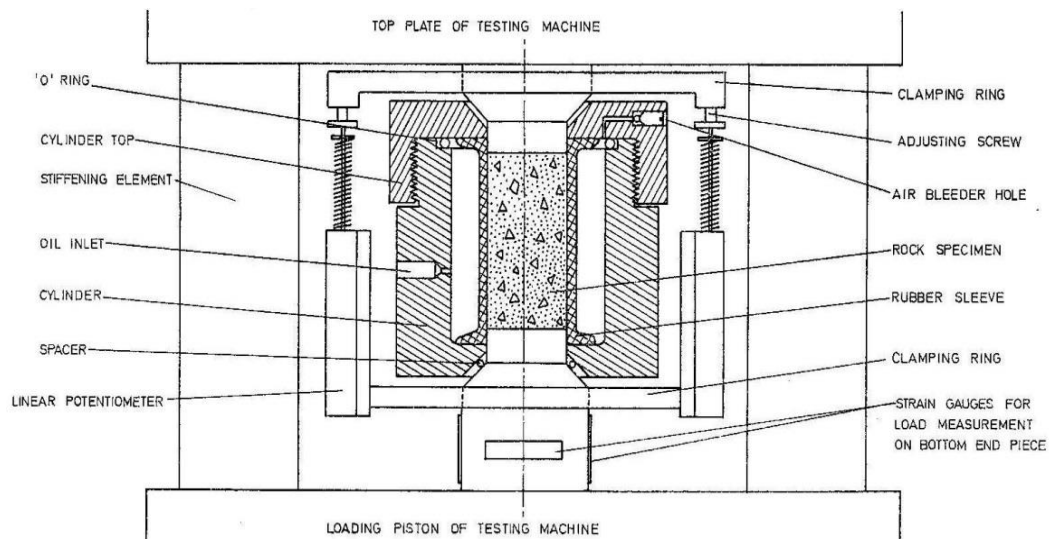


Figure 2.4 *Experimental set-up using steel columns for stiffening of testing machine (after Bieniawski et al, 1969)*

The set-up shown in Figure 2.4 is for triaxial tests. However the set-up for uniaxial tests is very similar. An oversized press with 5000 kN capacity was

used to ensure large enough forces to deform the columns. The equipment produced valuable information, an example is shown in Figure 2.18 in the section on load cycling. The practical problems were, however, that both load and deformations had to be measured by specially designed transducers. A special load cell had to be used in direct series with the specimen because most of the total applied load was eventually taken up by the stiffening columns. The load cell and the loading platens also deformed and hence the deformation transducers had to be as close as practically possible to the test specimen ends.

The length and cross-sectional areas of the columns had to be carefully designed so that their deformation would not require too much force, yet that they were able to absorb the excess energy stored in the system once the strength of the rock specimens passed the peak. A practical procedure was to leave a gap above the columns. The gap had to be just less than the expected deformation of the test specimen at the point of strength failure. From this point onwards the columns were deformed together with the specimen and were able to absorb any excess elastic energy. By making use of the columns the loading system was thus effectively stiffened.

Stavrogin and Tarasov (1995 and 2001) have also described ingenious high stiffness testing systems and have reported the results of many successful tests.

Although the use of stiff testing equipment provided information on the stress-strain behaviour of rocks in the post-peak region, it was still not always possible to trace the complete deformation behaviour. By making use of loading and rapid unloading cycles Wawersik (1968) succeeded in establishing the locus of curves in the post-peak region. He observed that the slope of the post-peak curves may be either negative or positive and classified the two types of failure as either Class I or Class II behaviour as is illustrated in Figure 2.5 below.

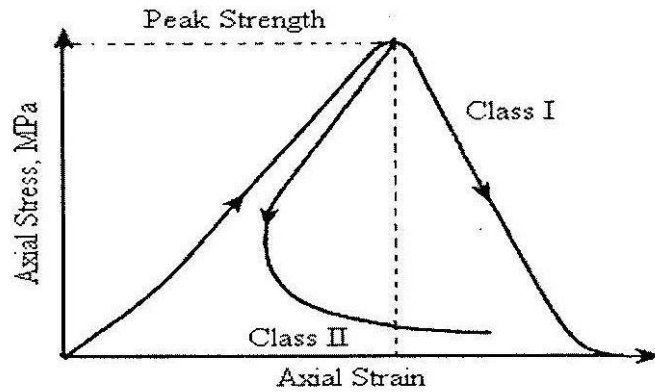


Figure 2.5 *Illustration of Class I and Class II behaviour of rock failure.*
(Adapted from ISRM 2007d)

The practical implication of Wawersik's observation is that, even with an infinitely stiff testing machine, it would still not be possible to trace the full stress-strain curves for rocks with Class II behaviour due to the elastic energy stored in the rock specimens. The concept is explained in detail in Section 2.3.3.

2.2 Stress-strain behaviour of rock in the post-peak region

Typical stress-strain curves for a number of rock types were published by Wawersik and Fairhurst (1970) and are reproduced below as Figure 2.6. As mentioned above the curves were obtained by load cycling,

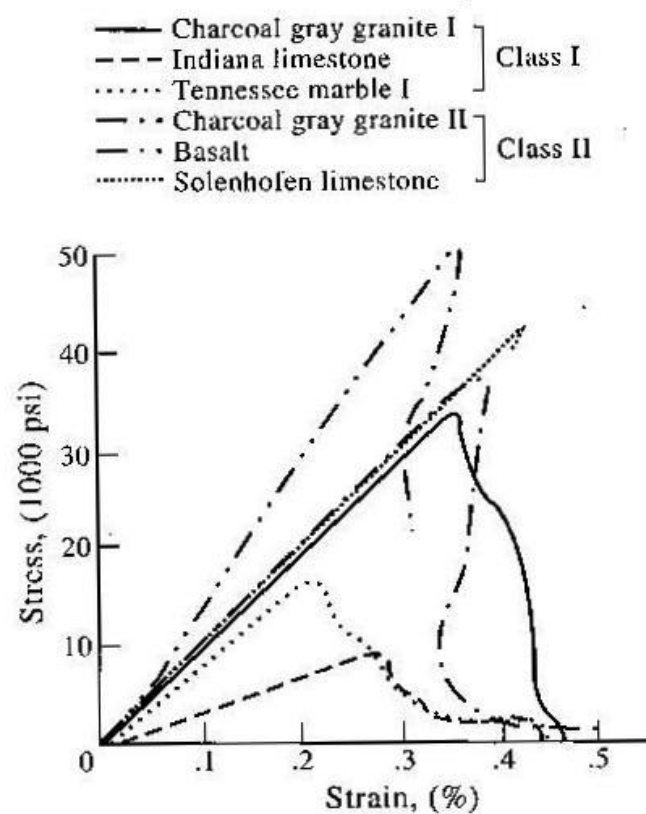


Figure 2.6 Complete stress-strain curves for six different types of rock
(after Wawersik and Fairhurst, 1970)

From the figure it may be seen that of the six types of rocks tested, three showed Class I and three Class II behaviour.

On the observed Class II type behaviour Wawersik and Fairhurst (1970) have commented: “... failure for Class II rock behaviour is self-sustaining, i.e. the elastic strain energy stored in the sample when the applied stress equals the compressive strength is sufficient to maintain fracture propagation until the specimen has lost virtually all strength” and also: “Obviously, Class II behaviour cannot be established by way of stiffening the testing machine alone because fracture would proceed even if the stiffness were infinite”.

2.3 Implications of Class I and Class II behaviour

Class I behaviour and Class II behaviour differ fundamentally and have to be discussed separately. This will be done in Sections 2.3.2 and 2.3.3 by means of typical curves. As the cause of the different behaviour is the quantity of elastic energy stored in the test specimens, energy and work will be briefly discussed in the next section.

2.3.1 Energy and work

Energy and work are quantified by Joules where one Joule is the work done by a force of 1 Newton resulting in a displacement of 1 meter. For example, a force of 10 Newtons (N) causing a displacement of 0.005 meters results in work done of 0.05 Newton-meters or Joules. Energy is the capacity to perform work and thus has the same units.

When a load-deformation graph is taken, whether linear or otherwise, the area under the graph is equal to the work done in Joules, provided the load is in Newtons and the displacement in meters. The principle is illustrated in Figure 2.7. For sake of simplicity a linear relationship is assumed. The energy or work may be calculated as follows:

$$\text{Energy or work} = \frac{1}{2} \sum_{i=1}^n ((F_i + F_{i+1}) \times (D_{i+1} - D_1))$$

Where: F is the force in Newtons

D is the displacement in meters.

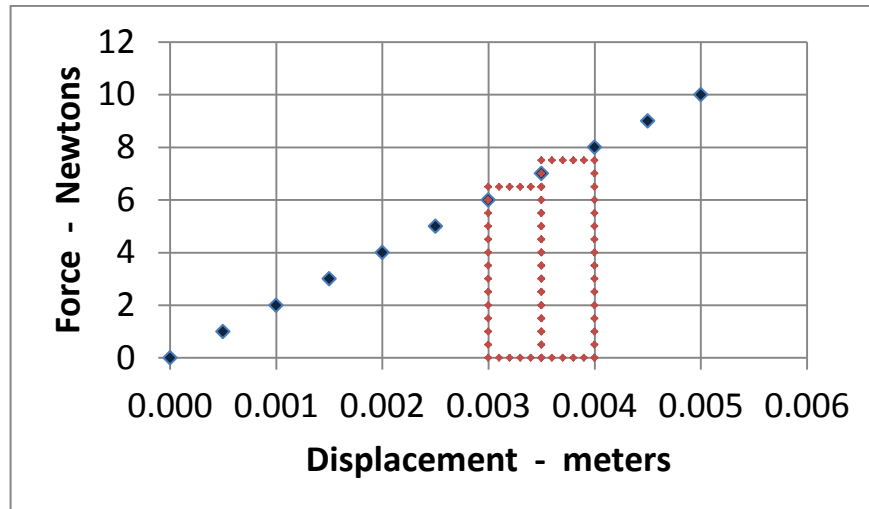


Figure 2.7 Principle of calculating work from experimental data

The total area may be calculated from real experimental data, even if the relationship is not linear.

Similarly, the units for the area under a stress-strain curve, representing strain energy density, will be Joules per cubic meter. This is because:

$$\text{Stress} = \text{Load/Area (N/m}^2\text{)}$$

$$\text{Strain} = \text{Change in length / original length (m/m)}$$

Thus the units for the product Stress $\times$ Strain are (N/m²) $\times$ (m/m) or J/m³.

2.3.2 Discussion of the implications of Class I behaviour

Typical Class I behaviour is illustrated in Figure 2.8. For reasons of simplification the elastic modulus is taken as a constant i.e. linear stress-strain relationship has been assumed. It must be noted that the descriptions below describe the behaviour of the rock only, regardless of the stiffness of the testing machine and hence regardless of the energy stored in the testing equipment. Whether the behaviour is of Class I or Class II mode, the lateral strains, not shown

on the explanatory diagrams, increase monotonically during the whole deformation cycle.

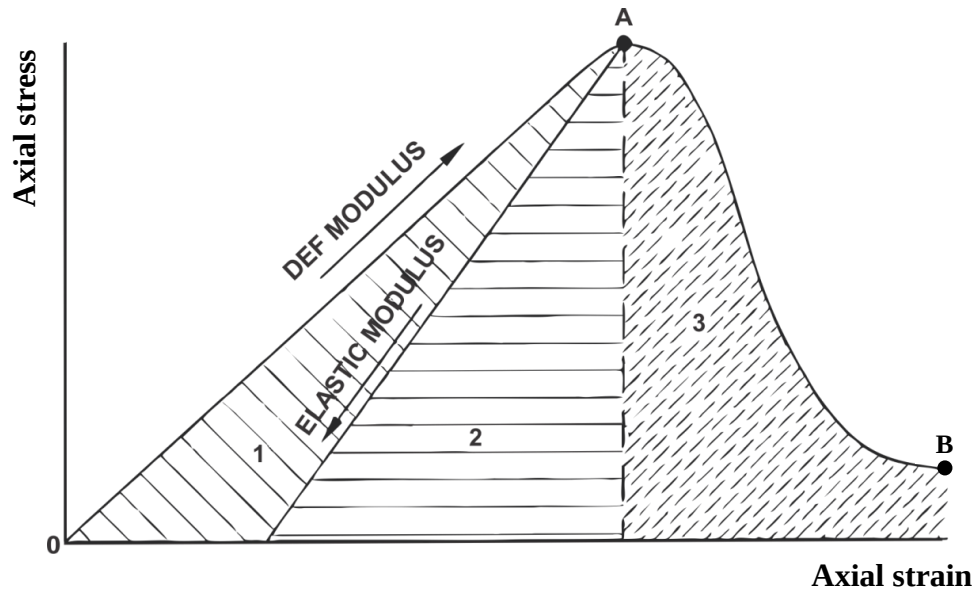


Figure 2.8 *Class I behaviour (adapted from Melis, 2011)*

The above curve from O to A to B illustrates typical Class I deformation behaviour. When testing such rocks, energy must be provided to the specimen continuously. When using testing equipment with insufficient stiffness i.e. machines accumulating elastic strain energy in their components, this elastic strain energy may exceed the energy required by the rock specimen and violent rupture will occur. However, in sufficiently stiff testing machines the elastic strain energy stored will be less than the energy required by the test specimen. The system (test specimen plus machine) is then stable and additional energy must be supplied to continue testing from A to B. Testing machines with sufficient stiffness thus allow tracing the post peak behaviour of rocks exhibiting Class I failure mode successfully. The significance of the shaded areas in the figure is as follows:

Areas 1+2+3	Total energy to be supplied by the testing machine to take test specimen from O to B.
Areas 1 + 2	Energy supplied to test specimen by testing machine at the point of peak strength (point A).
Area 1	Non-recoverable energy used up by test specimen during loading, e.g. energy required for plastic deformation, crack formation and extension, heat.
Area 2	Elastic energy available in test specimen at point A.
Area 3	Additional energy to be supplied to specimen by testing machine to continue from A to B.

During typical Class I behaviour, the axial strain increases from O to C monotonically i.e. all the way from initial application of load to the end of the test.

2.3.3 Discussion of the implications of Class II behaviour

Typical Class II behaviour is illustrated in Figure 2.9.

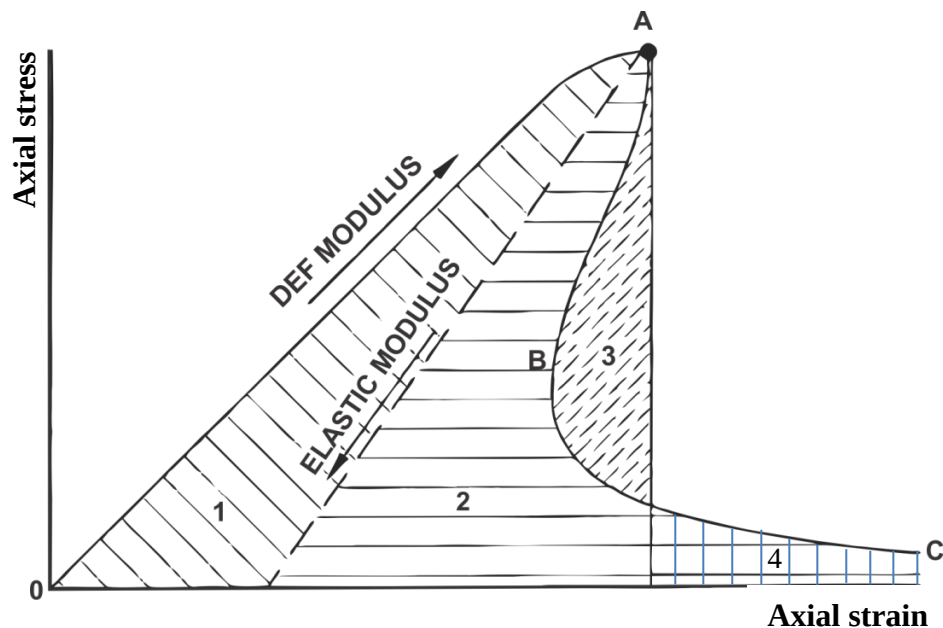


Figure 2.9 Class II behaviour (adapted from Melis, 2011)

During Class II behaviour, the axial strain increases from O to A, decreases from A to B and increases again between B and C.

As mentioned above when describing Class I behaviour, it must be noted that the descriptions below describe the behaviour of the rock only, regardless of the stiffness of the testing machine and hence regardless of the energy stored in the testing equipment. When testing rock having Class II behaviour energy must be provided to the specimen up to point A. Thereafter some of the energy must be withdrawn. Later again, energy must be supplied to continue the deformation process. The energy manipulation is explained by means of the shaded areas in Figure 2.9 as follows:

- | | |
|----------------|--|
| Area 1 + 2 + 4 | Total energy to be supplied by testing machine to take test specimen from O to C. |
| Area 1 + 2 + 3 | Energy supplied to test specimen by testing machine at the point of peak strength i.e. point A. |
| Area 1 | Non-recoverable energy used up by test specimen during loading, e.g. energy required for plastic deformation, crack formation and extension, heat. (Same as for Type I). |
| Area 2 + 3 | Elastic energy available in the test specimen at point A. |
| Area 2 + 4 | Energy required to continue from A to C |
| Area 3 | Excess elastic energy stored in the test specimen i.e. energy available in the specimen but not required for further deformation. In order to allow controlled testing from A to C i.e. to avoid instability and uncontrolled, violent failure, this surplus elastic energy has to be rapidly removed by the testing machine. If the rate at which energy can be released is fast enough then the specimen will fail in a controlled fashion; if not, violent failure will take place. |

When testing rock specimens with Class II type behaviour it is essential that excess elastic energy can be rapidly extracted from the test specimen in addition to the energy stored in the testing equipment. The combination of test specimen and test equipment is usually referred to as test system. The elastic energy stored in the test equipment can be minimized by designing the equipment as stiff as practically possible. In addition, by using servo-controlled testing equipment elastic energy can be released at fast rates.

2.4 The introduction of servo-controlled testing machines

Further development from Wawersik's work required a new approach to the testing of the post-peak behaviour of rock. Hudson and Harrison (1997) commented: *"Following the pioneering work by Cook, Bieniawski, Fairhurst and Wawersik in the late 1960s, it was realized that such stiff testing machines are inherently cumbersome and functionally inflexible. This led to the introduction of **servo-controlled** testing machines for obtaining the complete stress-strain curve for rock. It is important to note that the means by which a servo-controlled testing machine is able to follow the post-peak curve is different in principle and implementation from the stiff testing machines"*.

The development of stiff, servo-controlled testing machines has greatly improved the chances of successfully investigating Class II behaviour of brittle rock materials in the post-strength failure region of the stress-strain curves.

Data acquisition and control are handled by computers with the appropriate software. The feedback signals from transducers monitoring the specimen behaviour are continuously compared with the required testing program and any deviations are addressed by the signals sent to the rapid response servo valve system.

A schematic diagram of such a system is shown in Figure 2.10 below. The diagram shows the system as developed by the MTS Systems Corporation and has been copied from the ISRM Suggested Method (SM) (ISRM, 2007d). The ISRM SM will be further discussed in Section 2.5.

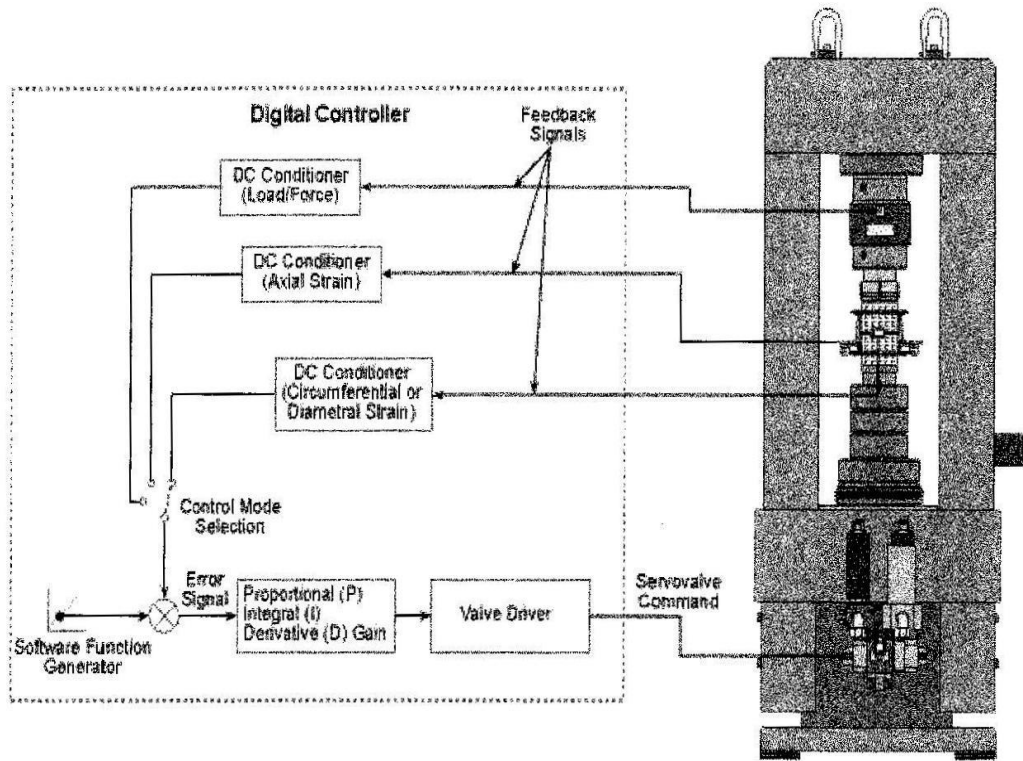


Figure 2.10 *Principle of operation of a closed loop testing system (after ISRM, 2007d)*

The positioning of axial and circumferential transducers is shown in Figure 2.11.

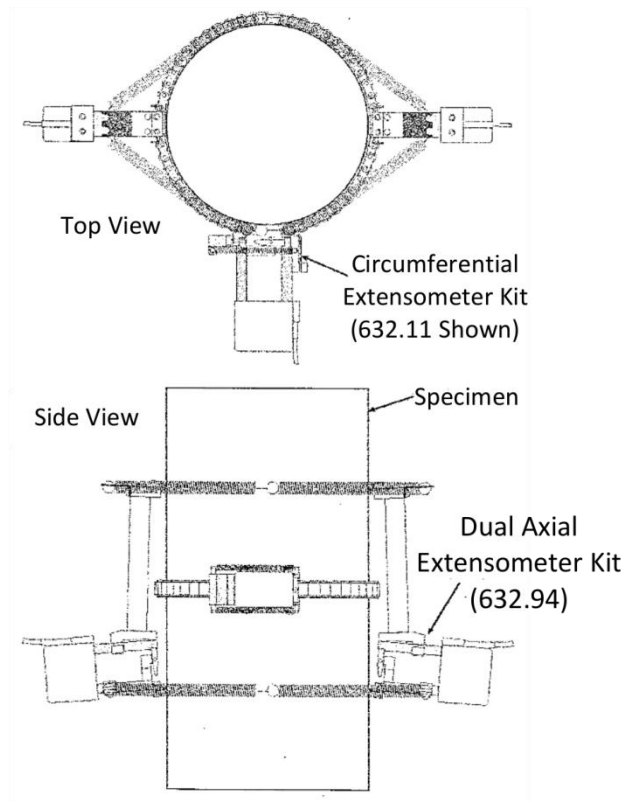


Figure 2.11 *Positioning of axial and circumferential transducers (after ISRM, 2007d)*

2.4.1 Methods of controlling testing machines

Before the introduction of servo-controlled testing machines it was customary to apply load to test specimens at either a controlled loading rate or at a controlled deformation rate. Since the introduction of computerized servo-controlled equipment it was possible to make use of additional variables, such as quantities measured during the tests, as feedback signals for programming further progress of the tests. Most researchers have been using the circumferential strain for controlling the closed-loop testing machines. The reason for this is that the circumferential strain increases monotonically regardless of whether the specimen behaviour is Class I or Class II. In fact, this is also the recommended method as proposed by the ISRM SM (ISRM, 2007d).

A notable exception is the control method described by Okubo and Nishimatsu (1985) and again by Okubo et al (1990). The method is called: “*Linear Combination of Stress and Strain as the Control Variable*”. They had noted that accurate measurements of lateral strains were difficult to obtain due to local failure e.g. spalling and therefore used the alternative approach which is applicable to rocks with Class I as well as with Class II behaviour (Okubo et al, 1990), (He et al, 1990). The control method has even been used successfully for tensile tests (Okubo and Fukui, 1996). Details of the method as described by Okubo and Nishimatsu (1985) are:

An estimated value for elastic axial strain (ϵ_e) is calculated by

$$\epsilon_e = \sigma / E'$$

where σ is the measured stress and E' is an assumed constant value for the elastic modulus.

The estimated elastic strain (ϵ_e) is used to adjust the measured axial strain (ϵ) as follows:

$$\epsilon - \epsilon_e = \epsilon - (\sigma / E')$$

The combined strain variable is then programmed to increase at a chosen constant time rate, typically 10^5 strain units per second.

The principle of linear combination of stress and strain as control variable is illustrated in Figure 2.12. The time intervals from t_0 to t_1 , from t_1 to t_2 etc. is constant. For comparison, Figure 2.13 illustrates the principle of simple “axial strain control”. However, the latter control method is only suitable for rocks with Class I behaviour.

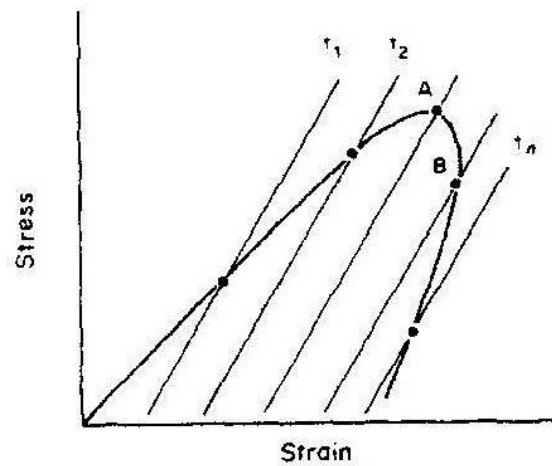


Figure 2.12 *Principle of the linear combination of stress and strain as feedback signal for programming the testing machine (after Okubo and Nishimatsu, 1985)*

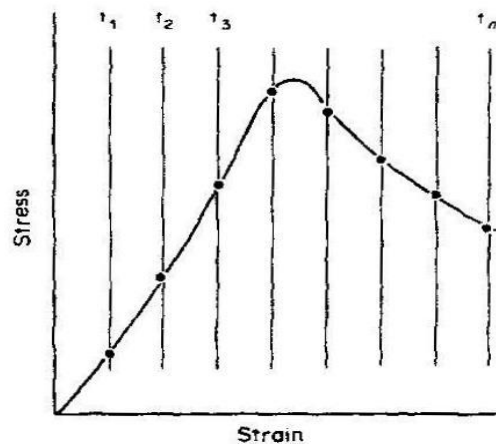


Figure 2.13 *Principle of linear control of axial strain as feedback signal for programming the testing machine (after Okubo and Nishimatsu, 1985)*

Okubo and Nishimatsu (1985) noted that: “... By this control method, failure of Class II rock was well controlled and stress-strain characteristics up to strength failure coincide with those obtained by the constant strain rate method. Of course, Class I rock can also be controlled by this method. It can be said that the

proposed method is an improvement compared to the constant strain rate method which is the standard method for rock testing”.

Using this method of a linear combination of axial stress and strain has the great advantage that a circumferential strain transducer is not required. When testing very brittle rock it is often not possible to prevent violent failure of the specimen and circumferential transducers are particularly vulnerable. The author witnessed a test on a brittle Quartzite with UCS of about 400MPa, performed at the Technical University of Freiberg in Germany, using MTS equipment. Failure was so violent that the expensive circumferential transducer was damaged, causing weeks of delay.

The application of the Okubo and Nishimatsu (1985) method should be further investigated for South African rocks. A drawback of the method is, however, that no information on the circumferential deformations can be obtained unless an additional circumferential transducer is installed.

2.5 The ISRM standard procedure for determining the post-peak behaviour of rock materials

The experience obtained by testing with servo-controlled testing systems allowed Fairhurst and Hudson (1999) to prepare a “Draft ISRM suggested method for the complete stress-strain curve for intact rock in uniaxial compression” (ISRM, 2007d). The draft has meanwhile been accepted fully as a ‘Suggested Method’ (ISRM, 2010).

The purpose of the ISRM Suggested Method as noted in its introduction is: *“The complete force-displacement curve for an intact rock specimen, whether tested in uniaxial compression or in a confined state, is useful in understanding the total process of specimen deformation, cracking and eventual disintegration, and can provide insight into the potential in situ rock mass behaviour”.*

The SM furthermore states: *“The term ‘force’ and ‘load’ are synonymous in this context, as are the terms ‘displacement’ and ‘deformation’. Thus a force-displacement curve is the same as a load-deformation curve. When the measured force is scaled by the original specimen area and the measured displacement is scaled by the original specimen length, a nominal stress-strain curve can be plotted”*.

The statement is perfectly correct and applicable to the pre-peak behaviour. In the post-peak region of the curves, the process of this ‘scaling’ results in effectively averaging the behaviour of the intact part of the specimen with the discrete movements along failure planes. Whereas this procedure is thus suspect, most reported literature follows the practice. As a matter of fact, this is the only really practical way of presenting the outcome of such post-peak tests and thus it will also be followed throughout this dissertation.

Unfortunately, even the methods and equipment recommended in the SM cannot guarantee a successful test. In Section 2.2 of the SM it is stated: *“The SM is intended for the characterization of intact rock. However, certain extremely brittle specimens, even under the most favorable testing conditions (e.g. stiff testing frame, slow loading rate, circumferential strain feed-back, low length/diameter ratio), may fail abruptly or even explosively when tested without confinement. It is suggested that these specimens may not be conducive to post-peak testing in a uniaxial mode, and should be tested in a confined state”* (ISRM, 2007d).

Somehow the above paragraph admits that, for extremely brittle rock, the available techniques are not sufficient to ensure a successful outcome of uniaxial tests. It is implied that certain tricks such as ‘confined state testing’ and low ‘length-to-diameter-ratio’ may be employed. In this context a successful test is one for which the full stress-strain curve can be traced. In effect the implication is that the post-peak behaviour and its determination are not fully understood. This statement has been confirmed by Kuipers (2011).

On post failure characteristics Napier (2009) admits that: *“the calibration of post-failure properties is a bit of guess work”* and for application of the results of such tests: *“Other numerical models use a simplified approach in which failing regions are represented by their assumed average stress-strain response; this is an empirical approach that needs careful checking and confirmation (typically absent)”*.

The consensus explanation for Class II behaviour of the rock material is the elastic energy stored in the unfailed parts of the test specimen. Ryder and Jager (2002) describe this phenomenon and its consequences aptly as follows:

“Certain rock materials, e.g. granite, exhibit even more extreme brittle behaviour: they can suffer virtually instantaneous, highly-localized failure and can even show positive post-failure slopes – so-called ‘Type II’ behaviour (Wawersik, 1968) ... Such behaviour, which is made possible only by strain release from the unfailed portions of the specimen itself after formation of an extremely localised failure surface, cannot be tracked even by an infinitely stiff testing machine: rather a fully servo-controlled machine has to be used.”

Although the use of servo-controlled test systems are indispensable for tracking stress-strain behaviour in the post-peak region of the curves the role of the stiffness of the equipment should not be underestimated and is discussed in the next section.

2.6 Stiffness of the test equipment

As was described in the previous sections the development of stiff, servo-controlled testing systems and an appropriate testing method made it potentially possible to determine post peak behaviour for Class II type rock materials. However, as quoted in Section 2.5, for extremely brittle rocks there is still no certainty of a successful test. The mechanical stiffness of the testing machine is still a very important factor because the excess elastic energy stored in both, equipment and specimen, must be relieved. The combination of stiff equipment

and fast acting servo-hydraulic control systems are therefore required to optimize the chance of successfully tracing the complete stress-strain curves.

According to Hooke's law the stiffness of an elastic body (K) is defined as the force (in Newtons) per unit displacement (in meters). A testing machine may be regarded as a combination of elastic spring elements. The machine thus reacts like a complex arrangement of springs in series and parallel. Typical components are not only the columns, cross bars, loading platens and spacers, but also the height of the column of hydraulic oil, hydraulic pipes, fittings and even the devices for measuring the hydraulic pressure. Manufacturers of testing equipment offer test frames with different heights. This is to provide the option of using triaxial test cells. The vertical testing space of the machine must then be adjusted depending on the height requirement of a particular type of test. This is done by means of spacers.

Theoretically it should be possible to calculate the stiffness for a given testing machine and the combination of all the above elements should result in a constant value for the stiffness.

However, from personal observations the author has noted that, as result of years of handling, the surfaces of, for example, the test machine spacers have developed minute ridges and dents. Such features as well as mismatches, for example surface flatness and non-parallelity, will influence the effective stiffness of the system and can even result in a non-constant value for the stiffness. Theoretical calculations of the stiffness can therefore, at best, only be approximations. A practical, experimental method for the determination of the actual mechanical stiffness of a testing machine is proposed in the following section.

2.6.1 Proposed practical method for the determination of the actual mechanical stiffness of a testing machine

The results of partially constrained swelling strain tests, that had been performed previously, have been reviewed specifically in the context of the research dealt with in this dissertation. An example of such a typical result has been documented in Appendix D. The conclusions reached are:

- During the swelling process the length of the rock test specimen gradually increases, but its increase is restricted by the resistance of the test apparatus.
- Due to the restriction offered by the equipment a swelling pressure develops.
- The force resulting from the swelling pressure acts against the equipment and causes it to deform.
- The deformation of the equipment allows the test specimen to swell further.
- Swelling of the specimen will continue until equilibrium is reached.
- The interaction between the force (resulting in stress) and the equipment deformation (resulting in strain) is proportional to the effective stiffness of the test equipment. Thus the slope of this relationship is a function of the stiffness of the equipment and is not the property of the rock tested.
- On the other hand, the final asymptotic values for the stress and the strain do represent the property of the rock.
- By calculating the load increment (ΔN) and the increment in deformation (Δm) from the measured stress and strain the stiffness of the test apparatus $K = \Delta N / \Delta m$ in Newton per meter can be calculated.

Considering the abovementioned observations on swelling tests it is now possible to critically look at creep tests and relaxation tests.

When performing creep tests, the load on a test specimen is kept constant while the deformation is monitored with time. A creep testing apparatus may thus be

regarded as infinitely soft. Should, on the other hand, a hypothetically, infinitely stiff apparatus be used for creep tests then, as soon as the test specimen deforms, the load would fall off to zero and the experiment would come to an immediate end. For relaxation tests, loads (or stresses) are applied to a desired stress level and thereafter no further (external) actions are taken by the test equipment. The relaxation strain with time is then monitored. An example of the observations during such a test by Bieniawski (1971) is presented in Figure 2.14.

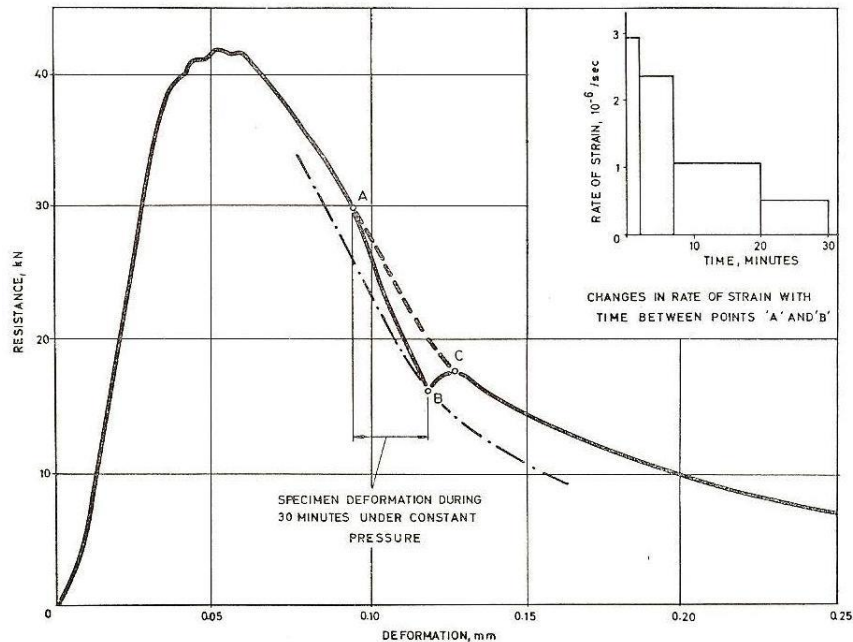


Figure 2.14 *Relaxation of Class I rock specimen (after Bieniawski, 1971)*

If it is possible to keep piston displacement constant, as is the case with servo-controlled testing machines, and as was the case with the test reported by Bieniawski (1971) using stiffening columns (Figure 2.4 of Section 2.1), then the relaxation behaviour can be used for calculating the actual mechanical stiffness of the testing apparatus. The reasoning behind the statement is as follows:

- Unless failure takes place, the force exerted on the specimen is always equal to the resistance force of the specimen.
- The deformation of the test specimen due to relaxation allows equal deformation of the testing apparatus.
- Deformation of the testing apparatus causes elastic rebound according to its stiffness and thus a reduction in the load applied to the specimen.
- The reduced applied load influences further relaxation.
- Relaxation of the specimen will continue until equilibrium is reached.
- The measured relaxation path is linear because it is dictated by the elastic behaviour of the apparatus.
- From the slope of the load-deformation curve the actual, mechanical stiffness of the testing apparatus can now be calculated.
- Also, from these observations it may be concluded that the rate of relaxation is dependent on the effective stiffness of the equipment used for the tests.

When scaling values from A to B in figure 2.14 the changes in load and deformation are 13.4 kN and 0.0243 mm resp. The calculated, actual stiffness of the equipment is therefore:

$$\begin{aligned}
 K &= \Delta N / \Delta m \\
 &= 13.4 \times 10^3 / 0.0243 \\
 &= 0.35 \text{ GN/m.}
 \end{aligned}$$

The proposed method for determining the stiffness of the equipment will work for Class I type behaviour tested in a relatively stiff testing machine. When testing material of Class II type behaviour, or, if the stiffness of the machine is insufficient for controlling Class I type behaviour, then instability of the test system will not allow the proposed method. Relaxation tests with the purpose of stiffness calculations may then be performed during the pre-peak loading phase of

the tests, regardless of Class I or Class II behaviour. If the specimen is stressed to above the level of ‘onset of unstable fracture propagation’ (see Figure 2.2 in Section 2.1), then sufficient data should be generated to perform the calculations.

2.7 Brittleness of rocks

A rock material may be considered brittle if, when subjected to stress, it will eventually break with no or little plastic deformation (strain). Brittle rocks thus absorb relatively little energy prior to failure. Breaking is violent and is usually accompanied by a loud noise. A ductile rock material can sustain relatively large permanent plastic deformations prior to failure.

Once the stress-strain curve of a material reaches its ‘level of failure’ there are various possible paths along which further deformation can take place. Figure 2.15 below shows the possibilities. In the diagram non-linear relationships are depicted because actual rock behaviour is generally non-linear.

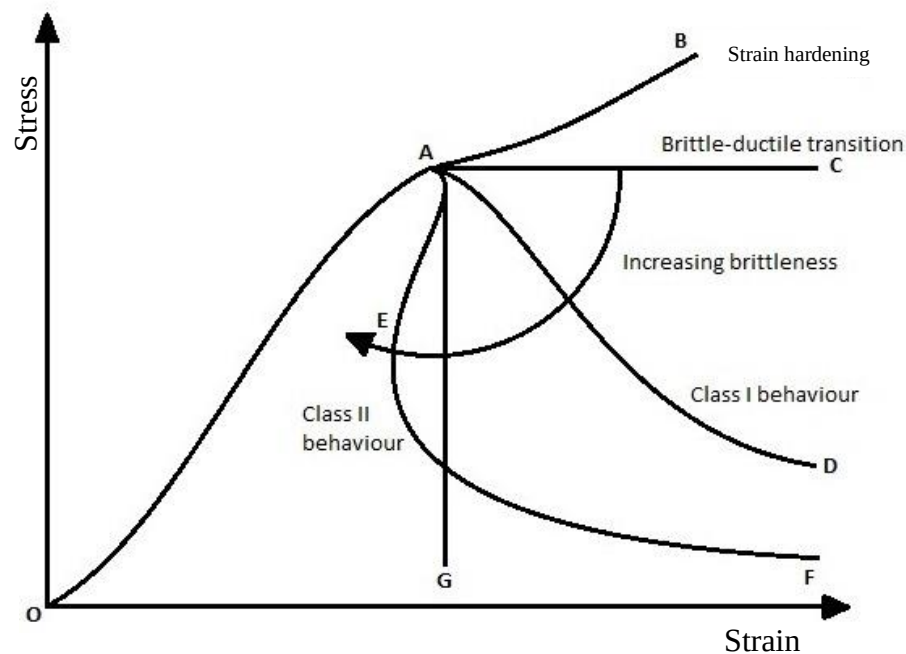


Figure 2.15 Schematic stress-strain curves for different modes of deformation (adapted after ISRM, 2007d)

Description of the possible load-deformation paths:

- Point A: The material fails. Failure may be strength failure i.e. when the maximum load bearing capacity is reached or when excessive deformations take place. .
- Line AB: Typical behaviour of a strain-hardening material. This type of behaviour is frequently observed for metals, but is not typical for rock except for some rocks when tested tri-axially with relatively high confining pressures.
- Line AC: Behaviour of a perfectly plastic material i.e. further strain takes place at a constant stress. The line represents the transition between brittle and-ductile behaviour.
- Line OAD: Typical Class I behaviour of rock.
- Line OAEF: Typical Class II behaviour of rock.
- Line AG separates Class I and Class II behaviour.

Brittleness increases from the brittle-ductile transition in a clockwise direction as indicated by the arced arrow. Rocks with Class II behaviour are thus far more brittle than rocks exhibiting Class I behaviour.

It must also be noted that for typical non-linear Class I as well Class II behaviour the brittleness is not a constant, but depends on the slope of the tangents of the curves in the post-peak region. It is thus extremely difficult to quantify ‘Brittleness’.

Tarasov (2008) has developed a “*Universal scale of brittleness for rocks failed at compression*”.

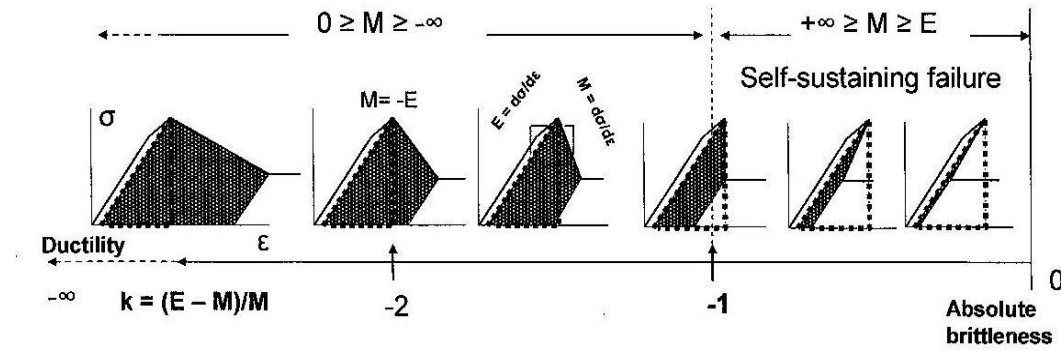


Figure 2.16 Universal scale of brittleness (after Tarasov, 2008)

The brittleness index (k) is calculated by $(E-M)/M$ where E is the elastic modulus and M is the post-peak modulus.

From left to right Tarasov's illustration shows post-peak curves with slopes progressively rotating in a clockwise direction. The shaded areas represent the energy required by the specimens. The illustrated behaviour gradually changes from Class I to Class II and shows how the energy requirements decrease. The values for M , calculated from the slopes, progressively change from 0 to $-\infty$ and further from $+\infty$ to the value of E . When $k = -1$, the available energy and the required energy are the same. When $M = E$ then $k = 0$. At this stage no further energy is required for failure and 'absolute brittleness' is reached.

The Tarasov index is thus a valuable index for assessing brittleness.

2.8 Review of published data

In this section some published data on test results is presented and discussed. Particular attention is given to the effect of experimental variables.

2.8.1 Results of uniaxial tests performed applying the Okubo and Nishimatsu (1985) control method

Okubo and Nishimatsu (1985) have published the results of successful tests conducted on eight types of rocks obtained by the new control method. The graphs are illustrated in Figure 2.17. All the rocks were tested in uniaxial compression. The test specimen diameters varied from 22 mm to 31 mm and the heights were between 42 mm and 62 mm. The height-to-diameter ratios were about 2.

Details for the rock materials tested:

- a) Akiyoshi marble. UCS = 90 MPa - Class I behaviour;
- b) Inada granite. UCS = 170 MPa - Class II behaviour;
- c) Izumi sandstone. UCS = 200 MPa - Class II behaviour;
- d) Kofu andesite. UCS = 190 MPa - Class II behaviour;
- e) Sanjome andesite. UCS = 95 MPa - Class II behaviour;
- f) Ogino tuff. UCS = 54 MPa - Class II behaviour;
- g) Kawazu tuff. UCS = 39 MPa - Class II behaviour
- h) Aoishi tuff. UCS = 34 MPa. - Class II behaviour.

The published results illustrate the successful application of the new control method (i.e. the linear combination of stress and strain). It is noteworthy and surprising that seven out of the eight types of rocks behaved in Class II mode. The marble is the only exception. Even the three low strength tuffs were Class II.

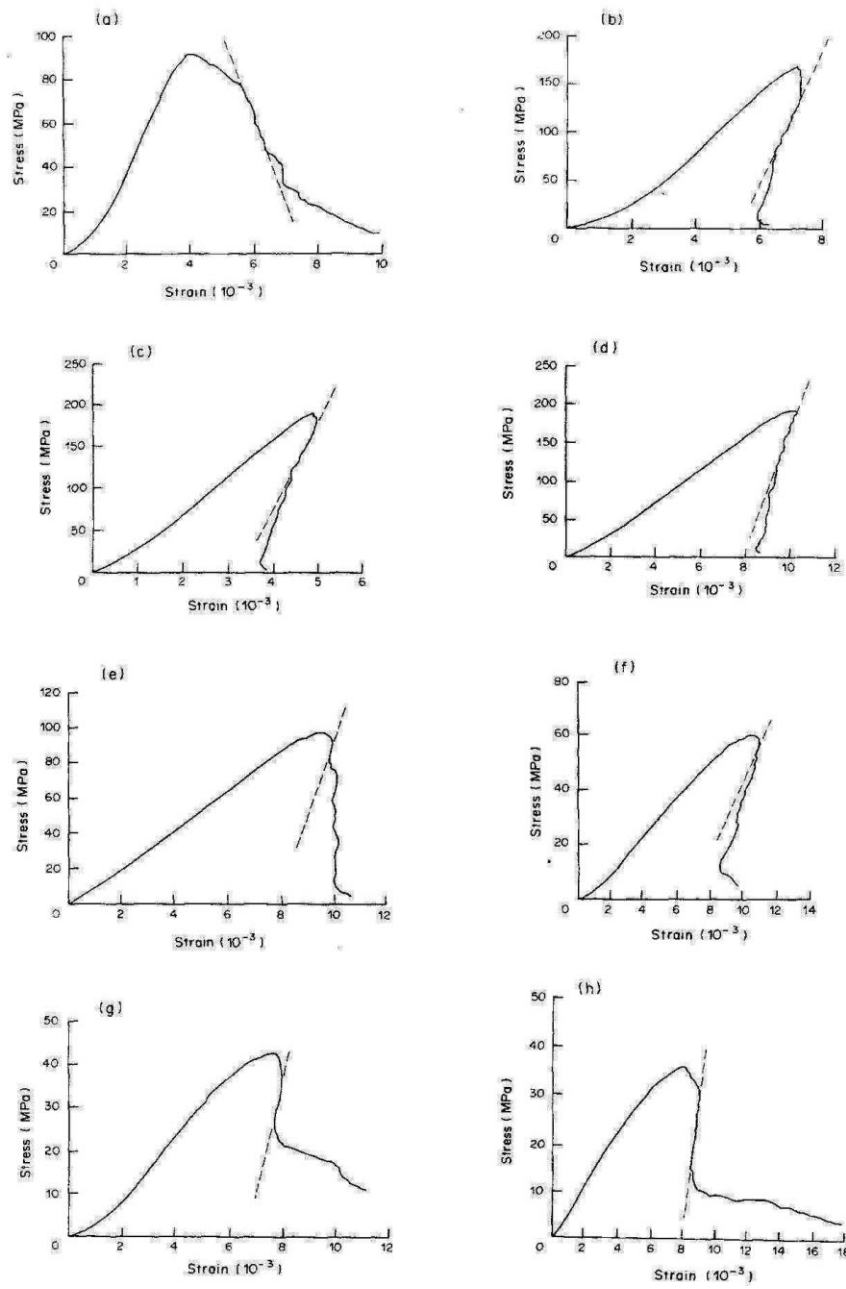


Figure 2.17 Complete stress-strain curves for eight rocks obtained by the new control method (after Okubo and Nishimatsu, 1985)

2.8.2 Load cycling

Load cycling, i.e. increasing the applied load followed by decreasing the load, is a commonly employed testing procedure. It allows differentiation between the ‘deformation modulus’ and the ‘elastic modulus’. ASTM D 4395 uses the term ‘recovery modulus’. Unless the material tested is perfectly elastic, the difference between the loading and unloading behaviour is a measure of the permanent deformation or of the plastic behaviour. Niandou et al (1997) have stated that:

“The analysis of the elastic behaviour is fundamental in constitutive modeling of material. In fact, the good knowledge of the elastic behaviour allows an accurate estimation of the plastic behaviour and damage in material. In general, elastic constants are determined from material responses during unloading paths.”

Bieniawski (1971) has compared curves for specimens tested continuously with curves for specimens tested using loading cycles. An example is shown in figure 2.18 below.

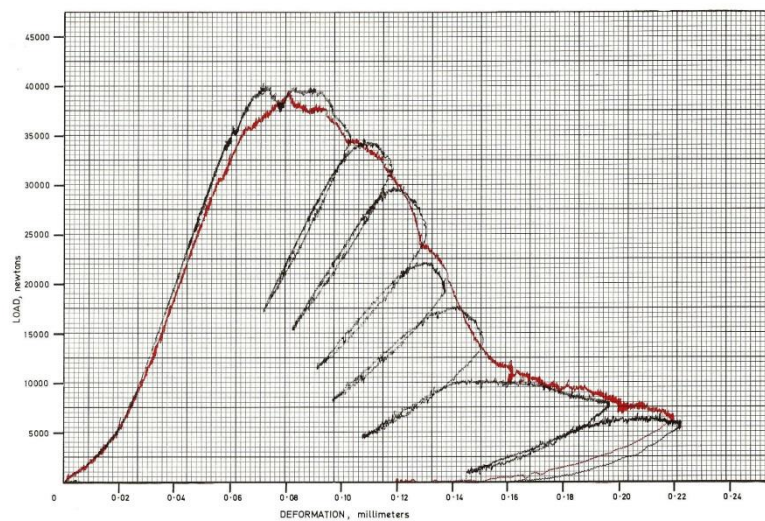


Figure 2.18 Comparison of load-deformation curves with and without cycling (after Bieniawski, 1971)

- It may be noted that the reloading curves practically join the curve for the control specimen tested continuously. Hence it is obvious that in effect the load deformation curves are not affected by cycling.
- The permanent, plastic deformation increases with subsequent loading cycles.
- The ‘elastic’ moduli on unloading as well as the ‘deformation’ moduli on reloading decrease with subsequent cycles and are lower than the deformation modulus for the first loading cycle. This is an indication of the progressive fragmentation of the specimen.
- The unloading/reloading loops display hysteresis properties.

Wawersik and Fairhurst (1970) have published very similar graphs leading to the same observations and conclusions.

When testing for the ‘in Situ’ deformability of rock both ISRM as well as the ASTM recommend cycling. Both standards are making use of the identical diagram. Figure 2.19 shows the diagram as copied from the ISRM SM (ISRM, 2007e)

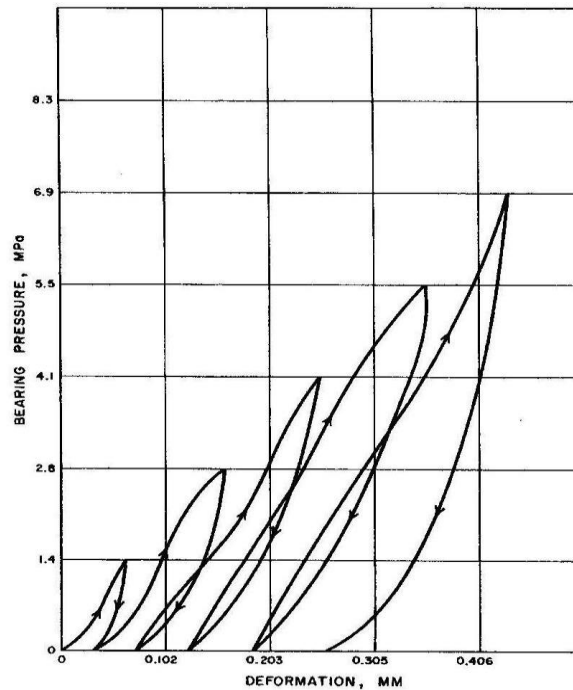


Figure 2.19 *Rock surface deformation as a function of bearing pressure (after ISRM, 2007e)*

When studying the above graph it may be seen that there is a significant difference in the slopes of the initial loading curve, the unloading curves and the reloading curves. This is to be expected when the tested material is not perfectly elastic.

However, it may also be noted that the V-shaped areas between successive peaks of the cycling curves are larger than the areas within the loops. This is particularly obvious from the open gap between the first unloading cycle and the second loading cycle. The implication of this observation is that, apparently, there is a significant saving in energy. (See previous section on Energy). It is mechanically impossible that the deformation of rock requires less energy when loading and unloading cycles are performed than when only one continuous loading cycle is done. Processes such as inelastic creep in the matrix material between the grains, closure of cracks, sliding between crack surfaces, formation of new cracks and extension of existing cracks require energy.

The author therefore suggests that there was hysteresis in the deformation measurement equipment when the above curves were obtained. This conclusion shows that it is important to understand the behaviour of the whole test system, including the loading and measuring equipment, when interpreting test results.

To obtain values for local secant modulus Fairhurst and Hudson, authors of the ISRM SM for determining the complete stress-strain curve for intact rock in uniaxial compression also recommend: “*Unloading in failure region*” (ISRM, 2007d). See figure 2.20 below.

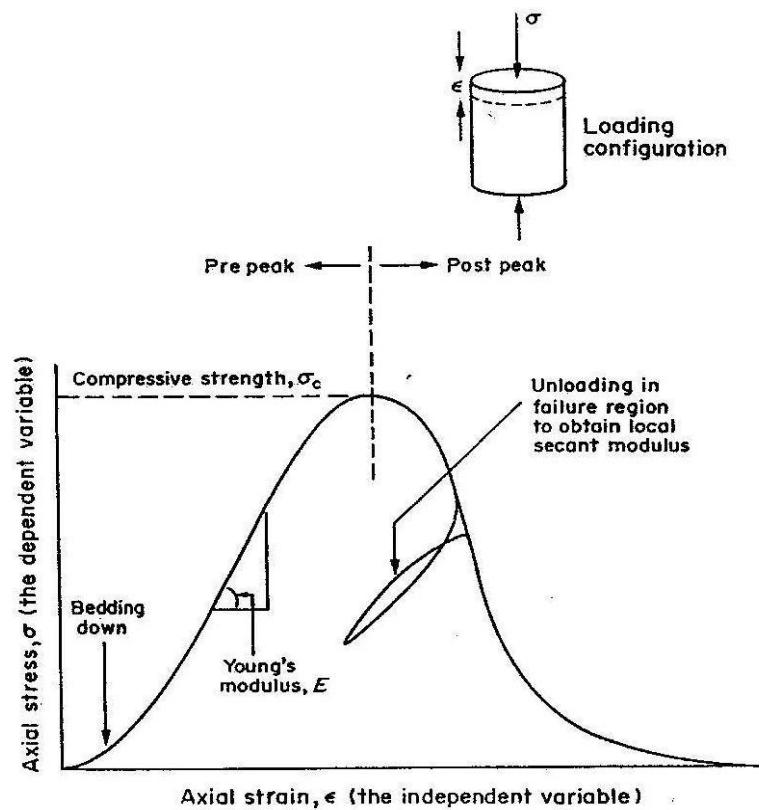


Figure 2.20 Complete stress-strain curve for a rock specimen showing the pre-peak Young’s modulus, compressive strength and post-peak modulus (after ISRM, 2007d)

Load cycling thus provides much additional information compared with the ‘standard tests’ for which one continuous loading cycle at a prescribed loading rate is recommended. In addition, careful interpretation of the results of cycling tests, whether in the laboratory or in-situ, can provide valuable information on the potential hysteresis behaviour of the measuring equipment and hence on the accuracy of the data reported.

He et al (1990) have studied the variation in unloading modulus by performing loading and unloading cycles in the post-peak region of the stress-strain curves obtained during uniaxial testing of five types of rocks. They have measured the “*effective unloading modulus*” at various stress levels. For purposes of comparison the values obtained for stress and for the effective unloading modulus were normalized by the corresponding values at the point of strength failure.

Copies of the results obtained for two of the materials are reproduced as

Figure 2.21.

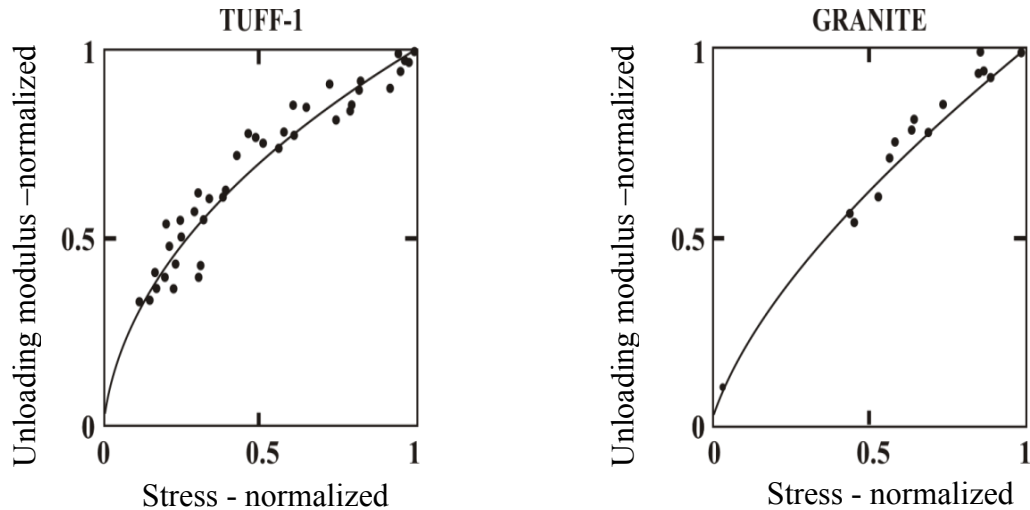


Figure 2.21 *Variation in effective unloading modulus in the post-failure region, tested in uniaxial compression (after He et al, 1990)*

In the post-failure region the resistance to load or stress is progressively decreasing. It was found that as the tests progress the effective unloading moduli decrease. From the experimental results He et al (1990) have concluded that:

“The important experimental result is that the elastic strain decreases continuously in the post-failure region regardless of rock types and confining pressure”. The results from He et al (1990), confirm the observations listed when discussing figure 2.18.

2.8.3 Influence of confining pressure

When testing rocks it is known that strength and deformation, as well as mode of failure (brittleness), are influenced by confining pressure. The post-peak behaviour is no exception.

Test results reported by Rummel and Fairhurst (1970) for a Tennessee marble with Class I type behaviour show clearly decreasing brittleness with increasing confining pressure (Figure 2.22). The post-peak moduli change gradually from a high negative value for the uniaxial test to lower negative values for increasing confining pressures and eventually even to a small positive value at the highest confining pressure of 55 MPa $\approx$ 40 % of the UCS. This indicates that in the latter case the material behaviour plots above the brittle–ductile transition and is even strain-hardening.

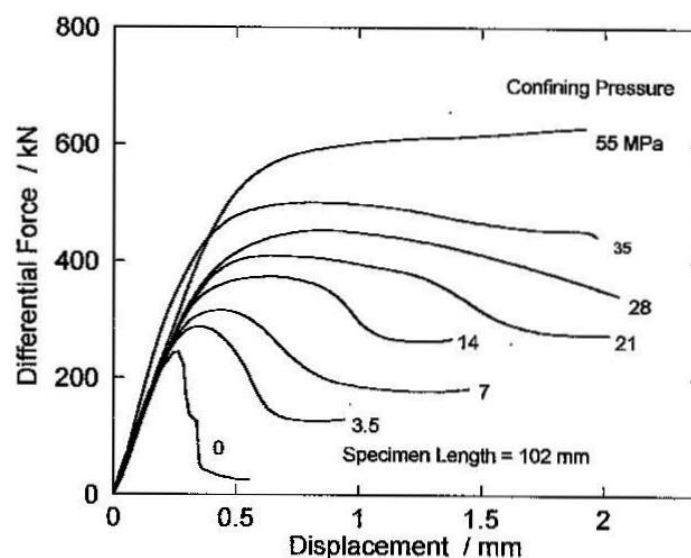


Figure 2.22 *The influence of confining pressure on post-failure behaviour of Tennessee marble (after Rummel and Fairhurst, 1970)*

Very similar results have been reported by Okubo et al (1990) for an Andesite with an UCS of approximately 100 MPa (maximum confining pressure 40 MPa $\approx$ 40 % UCS) and a Tuff with an UCS of approximately 34 MPa (maximum confining pressure 20 MPa $\approx$ 60 % UCS). See Figure 2.23 below. Both these rocks changed from Class II behaviour for uniaxial tests to Class I behaviour at higher confining pressures. The Tuff at the highest confining pressure once again shows strain-hardening tendencies.

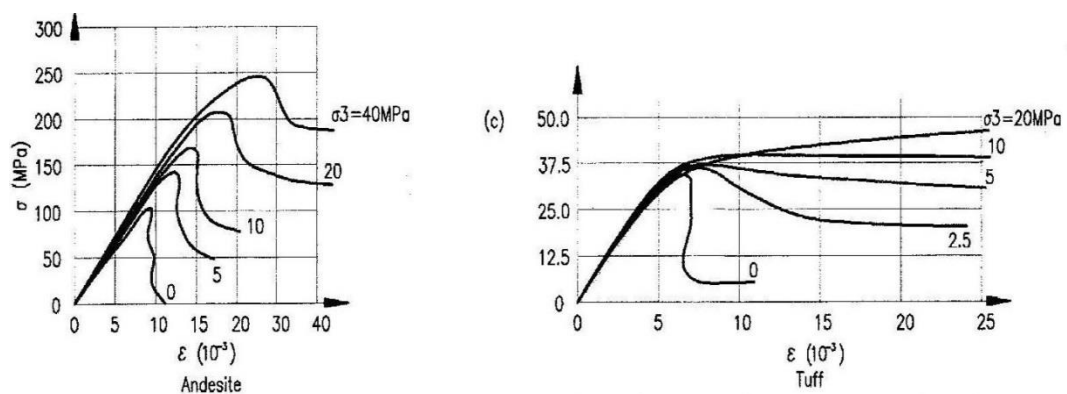


Figure 2.23 *Influence of confining pressure on post-failure behaviour of Andesite and Tuff (after Okubo et al, 1990)*

Stavrogin and Tarasov (2001) have reported the results of experiments on the effect of confining pressure on six types of rock. The rocks tested were Marble (Urals), Lignite (Shurabsk), Granite (Karelyia), Sandstone (NBP Donbass), Sandstone (BP Donbass) and Sulphide ore (Noril'sk). All the results showed the same tendencies as reported by Rummel and Fairhurst (1970) and Okubo et al (1990).

He et al (1990) have measured the elastic and non-elastic strains for a granite in the post-failure region when performing triaxial tests. Their graph for the Granite is shown in Figure 2.24.

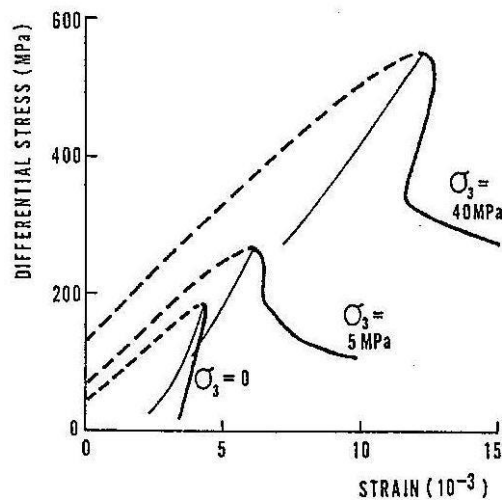


Figure 2.24 *Elastic and non-elastic strains on Granite in the post-failure region under confining pressure (after He et al, 1990)*

They concluded that: “In triaxial compression, it was found that the portion of non-elastic strain becomes larger than in uniaxial compression”. All three post-peak curves show Class II behaviour, yet it is notable that there is no definite trend in the slopes.

For the post-peak regions of stress-strain curves most published results show rotation of the slopes in anti-clockwise direction with increasing confining pressures and hence decrease in brittleness.

There are, however, some noteworthy exceptions. The results reported by Wawersik and Brace (1971) for a Westerly granite illustrated below in Figure 2.25 do not show the abovementioned trend. The UCS of the rock is about 260 MPa and the maximum confining pressure about 150 MPa ($\approx 58\%$ UCS). No obvious trend of decreasing brittleness with increasing confining pressure is demonstrated.

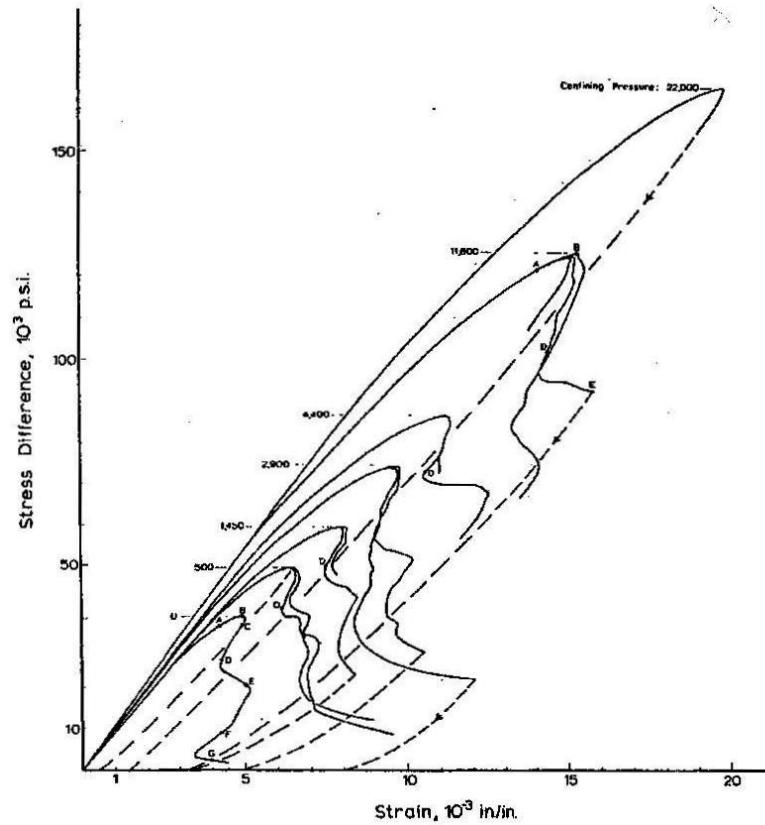


Figure 2.25 *The influence of confining pressure on the post-peak behaviour of Westerly granite (after Wawersik and Brace, 1971)*

Tarasov and Randolph (2008) even reported increase in brittleness with increasing confining pressures. The experiments were performed on a Dolerite with UCS of 309 MPa. The maximum confining pressure was 150 MPa ($\approx 49\%$ UCS). A frictionless shearing mechanism after fracture was proposed as explanation for the observed phenomenon.

It is interesting to note that the UCS of the rocks reported on by Wawersik and Brace (1971) and also by Tarasov and Randolph (2008) were both of very high strength according to the Deere Classification whereas most of the other types discussed in this section were of much lower strength.

2.8.4 Influence of size and shape

Labuz and Biolzi (1991) have studied the effect of height-to-diameter ratio by comparing the deformation behaviour of specimens with diameters of 26 mm and heights of 52 mm and 78 mm respectively. The tests were done on Indiana limestone. A confining pressure of 2 MPa was applied. They concluded that the response differs with an increasing tendency towards instability as height-to-diameters increase.

For illustrating the effect of ‘size’ they compared the behaviour of ‘notched’ specimens with height-to-diameter ratio 2 and diameters 26 mm and 52 mm respectively. For the 26 mm diameter specimens “stable” Class I type behaviour, while for the 52 mm diameter specimens Class II type (“*snap-back*”) response was reported.

They concluded that a size scale exists and that the behaviour demonstrates a “structural response” and not a material property and, in addition, that the same ‘softening’ material can appear with Class I as well as Class II stability behaviour. They also stated: *“In most cases the behaviour observed after peak load is a structural response and not a material characteristic”*. Labuz and Biolzi (1991) furthermore reached the conclusion: *“The post-peak behaviour of a structure composed of an elastic, softening material, such as most rock, is dependent upon the size and geometry of the global element. The basis of this observation is that the potential energy of the system, stored per unit of volume, is converted to fracture energy within the rock, dissipated per unit of area. Consequently, a natural size effect appears in the load-displacement response”*.

2.8.5 Influence of loading history

Many rocks creep when a constant stress is applied over time. It is thus to be expected that the loading history will have an influence on the outcome of tests.

Peng (1973) has reported on the influence of the deformation rate on the post-peak behaviour of an Arcose sandstone tested in uniaxial compression. The height and

diameter of the specimens were 64 mm and 32 mm respectively. As was to be expected, the strengths increased with increase in deformation rate. See Figure 2.26. The results show a decrease in brittleness with increase in deformation rate.

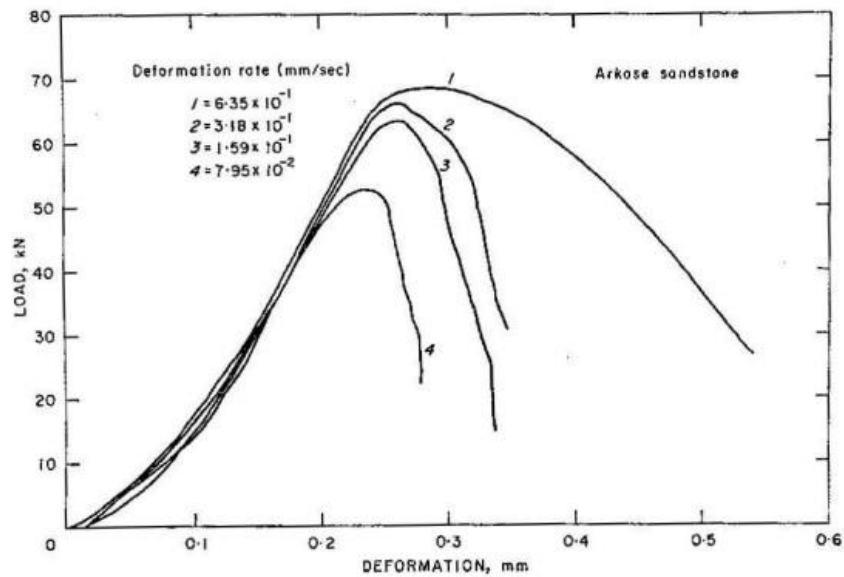


Figure 2.26 *The influence of deformation rate on post-failure behaviour of Arkose sandstone tested in uniaxial compressions (after Peng, 1973)*

Wawersik and Brace (1971) have studied the influence of loading history for a Westerly granite. Their findings are reproduced in Figure 2.27.

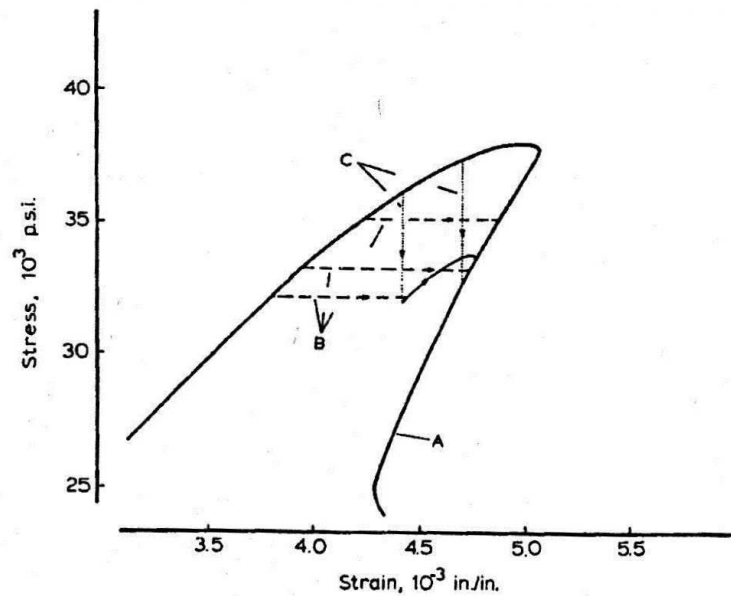


Figure 2.27 *Influence of loading history on uniaxial stress-strain behaviour of Westerly granite (after Wawersik and Brace, 1971)*

The conclusions reported are: “.. When sufficient creep had occurred that Curve A was reached, then further behaviour was identical with that of a specimen which had been taken to the ultimate strength. In other words, as soon as Curve A was reached the specimen either fractured violently or could be unloaded in the usual way to trace out the remainder of Curve A”. It may also be seen from the graph that the peak strength is lower for tests that allow time for creep to take place. Wawersik and Brace (1971) concluded that the eventual post-peak behaviour remains the same.

When investigating the effect of “stress relaxation” for marble, Hudson (1971) has reported that : “*When the strain is increased after a hold, the curve seems to rise to a continuation of the original curve, indicating that the descending part of the complete stress-strain curve is not affected by this type of variation in the strain increase*”.

The research results discussed in this section thus indicate that the loading history does not affect the stress-strain behaviour in the subsequent region of the curves.

2.8.6 Brief summary of information gathered

The main findings of the study of relevant literature may be summarized briefly as follows:

- It was described how equipment for successfully testing rocks in the post-peak region of the stress-strain curves has been developed during the last few decades.
- Examples of the stress-strain behaviour of rock in both pre-peak as well as post-peak regions have been documented.
- Class I and Class II failure modes and their implications have been described in detail.
- Stiff, servo-controlled testing equipment has been described.
- It has been shown that the method of controlling the testing machine may have an influence on whether the full post-peak behaviour can be traced.
- Confining pressure has a significant influence on the post-peak deformation behaviour, the strength and the mode of failure.
- It has been reported that ‘highly localized failure’ is the predominant mode of failure when successful tests on rocks with Class II type behaviour are performed.
- The elastic strain energy stored in the unfailed portions test specimens has been reported as the explanation for Class II behaviour.
- It has also been quoted from Section 2.2 of the ISRM SM that: *“The SM is intended for the characterization of intact rock. However, certain extremely brittle specimens, even under the most favorable testing conditions (e.g. stiff testing frame, slow loading rate, circumferential strain feed-back, low length/diameter ratio), may fail abruptly or even explosively when tested without confinement. It is suggested that these specimens may not be conducive to post-peak testing in a uniaxial mode,*

and should be tested in a confined state” (ISRM 2007d). This means that, even when all the variables are under control and when the best, modern testing equipment and techniques are employed, a successful test providing post-peak data can still not be guaranteed.

The above findings were considered when the testing programmes described in the following chapter were planned.

CHAPTER 3

EXPERIMENTAL PROGRAMME

In this chapter the experiments conducted for investigating the premise stated in Chapter 1 are documented.

From the information described in the previous chapter the conclusions were reached that:

- The post-peak strength and deformation behaviour of rock are not fully understood.
- The elastic energy stored in the unfailed portions of the test specimen is responsible for the slope of the post-peak curve.
- The post-peak behaviour is influenced by the test specimen geometry.

The aim of the experimental programme was therefore to prove that Class I or Class II behaviour of rock (during laboratory testing) is partially a test specimen phenomenon and not solely a characteristic of the rock material.

3.1 Method of investigation

The laboratory investigation was designed to test specimens of the same rock material, having the same diameters, but having different height-to- diameter ratios. It was expected that the contribution of the elastic response of the unfailed portions of the test specimen relative to the overall deformation response will result in significant differences in the post-failure behaviour. The slopes of post-failure curves were thus expected to be affected with the implication that apparently even the brittleness will be different.

It was predicted that, should the premise be correct, a number of consequences would follow:

- Class I or Class II behaviour is not solely a rock material characteristic.
- Class I or Class II behaviour depends on the relative influence of the ‘elastic’ and the ‘inelastic’ components of the test specimen.
- The practice of using the average post-peak stress-strain response is scientifically incorrect.
- The energy per unit volume of rock required for deforming rock specimens in the post-peak region is not constant.
- The apparent ‘brittleness’ is dependent on test specimen geometry.
- The following statement as published in the ‘introduction’ to the ISRM SM is not fundamentally correct for the post-peak part of the curves:
“The terms force and load are synonymous in this context, as are the terms displacement and deformation”.

3.1.1 Testing programme

For testing the premise, test specimens with different height-to-diameter ratios had to be tested. Furthermore it was essential that successful tests were performed to obtain the required information. A successful test for the

determination of post-peak properties is generally taken as a test during which the test data can be successfully captured from the start of testing to a point that is determined by the tester.

Ideally the premise should be tested for materials with Class I behaviour and Class II behaviour. The testing programme eventually adopted to obtain the required information consisted of three phases as follows:

- Testing a material with Class I type behaviour. A stabilized soil material was found which could be tested in an ordinary ‘soft’ testing machine.
- Selecting a suitable rock material which would be of medium to high strength and would behave in a Class II fashion. Anorthosite was considered to be suitable and a preliminary uniaxial test was conducted to confirm this.
- Performing the main series of tests on the abovementioned Anorthosite. Triaxial tests with a relatively low confining pressure were conducted.

3.2 Determination of post-peak behaviour of a stabilized soil material

During tests conducted at the CSIR’s Division for the Built Environment it was observed that stabilized soil materials exhibit Class I type behaviour in the post peak region of the stress-strain curves (Mgangira, 2008). Furthermore, it was possible to record the full stress-strain behaviour even when testing in a very ‘soft’ type of hydraulic testing machine while the applied loads were controlled manually (Coetzer, 2010). In addition, elastic recovery was observed on unloading at the end of the tests.

Material with such behaviour was considered suitable to perform tests to study the effect of test specimen geometry on the post-peak behaviour. A number of tests were therefore conducted.

3.2.1 Equipment for testing stabilized soil specimens

Suitable testing equipment was available and operational. In addition to the manually controlled Baldwin hydraulic testing machine with capacity of 300 kN, the test set-up included an accurate load cell, a LVDT type deformation transducer and computerized multichannel 'Datataker' data recording equipment. Figure 3.1 below shows the test set-up, excluding the data acquisition equipment.



Figure 3.1 *Experimental set-up for testing stabilized soil specimens*

3.2.2 Test specimens

A cement-stabilized soil of granitic origin was used for the tests. The soil/cement mixture was stamped manually into cylindrical moulds. The length of the moulds

allowed for ‘standard’ size and shape specimens with a diameter of approximately 102 mm and height-to-diameter ratio of approximately 1.15. Although the test specimens were ‘hand-made’, their densities were remarkably constant as may be seen in Table 3.1. The available moulds did not allow for the preparation of specimens with other height-to-diameter ratios. All the specimens for the current test programme were prepared on the same day.

3.2.3 Testing programme

Apart from testing four specimens with the standard height-to-diameter ratio of 1.1, a further specimen with height-to-diameter ratio of 2.3 was made up by simply using two ‘standard’ specimens, one on top of the other. Similarly, one additional specimen with height-to-diameter ratio of 3.4 was made up using three ‘standard’ specimens. All the tests were performed on the same day to ensure that all specimens had the same curing age.

3.2.4 Evaluation and presentation of results

The following data was captured in Excel spreadsheet format:

- Date and time of test (accurate to the nearest second).
- Voltage output of the load cell, from which the applied loads and hence stresses were calculated.
- Voltage output of the LVDT transducer, from which deformations and strains were calculated.

Stress-strain curves were plotted for the data obtained and are reproduced in Appendix A. All graphs were plotted to the same scales for easy comparison. Only the individual data points are shown. The points were not connected because the density of the points gives an indication whether the testing machine

was in perfect control or otherwise.

The ‘**Average**’ deformation moduli (see Figure 2.1) for the pre-peak region of the curves were calculated. Similarly, ‘**Average**’ values for the post peak deformation moduli were determined by selecting a linear region of the descending part of the curves. The ‘elastic’ moduli for the unloading curves of the cycles in the post-peak region of the stress-strain curves were determined as ‘Tangent’ values, calculated at the constant stress of 0.25 MPa. A typical stress- strain curve, and the method of determining the values for the moduli, is illustrated in Figure 3.2 below.

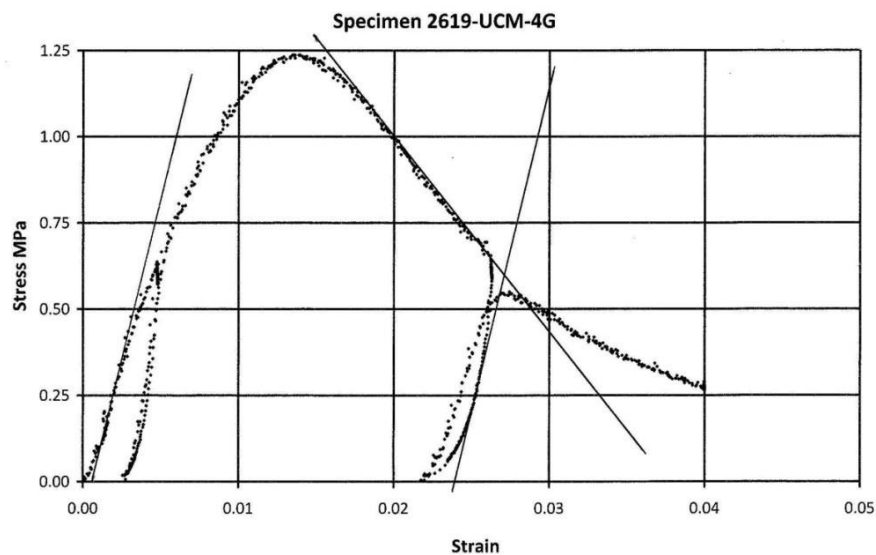


Figure 3.2 *Typical stress-strain curve illustrating method of determination of moduli.*

The results thus obtained are summarized in Table 3.1.

Table 3.1 Results of unconfined compression tests on stabilized soil

Test Specimen No	SPECIMEN DIMENSIONS					SPECIMEN TEST RESULTS				
	Dia.	Height	Height to Dia Ratio	Mass	Density	Strength (UCS)	Deformation Modulus			Tangent Elastic Modulus @ 0.25MPa on unloading
	(mm)	(mm)		(g)	(kg/m ³)	(MPa)	Pre-Peak	Pre-peak Modulus-Ratio	Post-Peak	(MPa)
2619-G	(mm)	(mm)		(g)	(kg/m ³)	(MPa)	(MPa)		(MPa)	(MPa)
1G	102.2	117.2	1.15	2077	2160	1.13	137	121	-63	na.
2G	101.8	117.1	1.15	2076	2180	0.99	66	67	-40	na.
3G	102.3	116.9	1.14	2072	2160	1.07	94	88	-56	188
4G	102.0	117.2	1.15	2074	2160	1.23	126	102	-57	210
5/6G	102.3	234.0	2.29	4147	2160	0.88	177	200	-124	250
7/8/9G	102.4	351.2	3.43	6222	2150	0.79	211	267	-122	311
5G	102.2	117.2	1.15	2073	2150	na	← Details of individual specimens before combining			
6G	102.3	116.8	1.14	2074	2160	na				
7G	102.3	117.3	1.15	2073	2150	na				
8G	102.8	117.0	1.14	2074	2140	na				
9G	102.1	116.9	1.15	2075	2170	na				

3.2.5 Discussion of results of tests on stabilized soil

In this section the term ‘elastic’ modulus is used for the slope of a curve during unloading. This is because, during unloading, the strain recovery as result of decrease in stress is elastic. Unloading should not be confused with the post-peak reduction of load, the latter being due to the decrease in load bearing capacity of the, by then, partially fractured specimen. The term ‘deformation’ modulus is used for the rest of the evaluations because in both, the pre-peak and the post-peak regions of the curve the non-elastic, i.e. non-recoverable strains, influence the slope of the curves.

During the tests the formation of predominantly vertical surface extension cracks was observed. Figure A7 of Appendix A shows an example of a specimen with such cracks. In Figure A8 a photograph of a specimen, after carefully removing the loose fragments, is reproduced. The fragments resulting from the formation of the cracks caused a reduction of the effective cross-sectional areas of the tested specimens which have affected the stresses and thus also the strains and hence the values for the calculated deformation moduli. The outcomes of the tests were thus affected, but this does not invalidate the conclusions reached below.

Strength

The strength is taken as the maximum load bearing capacity of the test specimens and is thus the stress at the peak of the stress-strain curve. Bieniawski (1967b) named this peak value the point of “strength failure”. Stavrogin and Tarasov (2001) are using the terms pre-failure and post-failure when discussing specimen behaviour. In this dissertation the terms pre-peak and post-peak will be used.

Figure 3.3 shows strength plotted against height-to-diameter ratio.

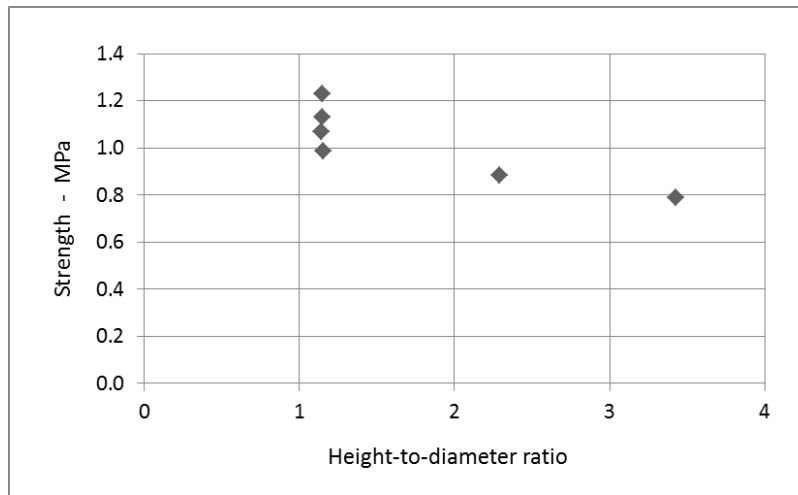


Figure 3.3 *Peak strength vs height-to-diameter ratio for stabilized soil*

The following may be observed from the above figure:-

- The strengths for the four ‘standard’ size specimens i.e. the specimens with height-to-diameter = 1.1 vary from 0.99 MPa to 1.23 MPa, i.e. the average strength is 1.10 MPa and the range is 0.24 MPa ($\approx 22\%$). The variation is reasonable considering the type of soil-like material and the fact that the specimens were made manually. The variation in values is, however, unfortunate because only one specimen could be tested for each of the other two height-to-diameter ratios. Statistical curve fitting to the results of so few tests has thus not been done and only the trends were investigated.

- In rock mechanics terms these strengths are extremely low. However, it is exactly this low strength material that made it possible to obtain records of complete stress-strain curves using relatively simple testing equipment and a very ‘soft’ type of testing machine.
- The strength shows a clear decrease with increasing height-to-diameter ratios. This was to be expected and is in line with findings for rock. (John, 1972), (Mogi, 1966). For testing of rocks it is commonly accepted that the strength is levelling off around a height-to-diameter ratio of 2.5. Therefore, for standard rock testing ASTM D7012 recommends testing specimens of between 2.0 and 2.5 while the ISRM (2007a) recommends this ratio to be between 2.5 and 3.0. In South Africa most testing laboratories aim for a ratio of 2.5, thus attempting to satisfy both recommendations.

Pre-peak deformation modulus

As mentioned above, the deformation modulus has been taken as the ‘average’ slope of the stress-strain curve in the pre-peak region i.e. loading region. ‘Average’ in this context has been defined in the recommendations by ASTM and ISRM. Figure 3.4 below shows the ‘average’ deformation modulus plotted against height-to-diameter ratio.

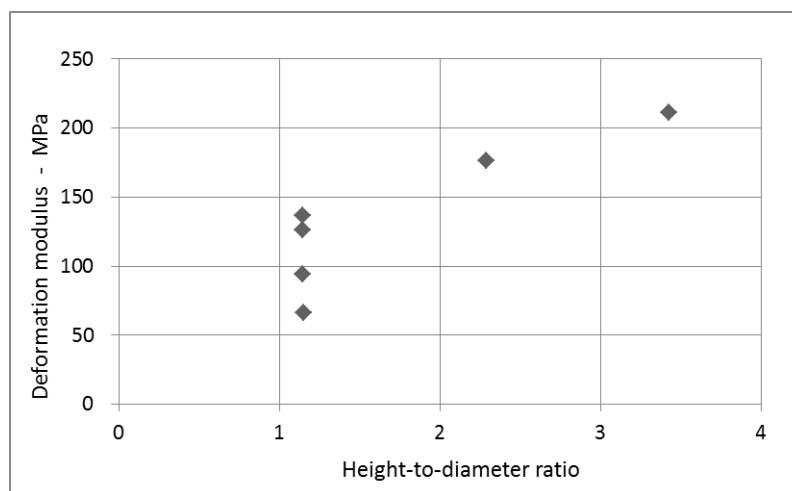


Figure 3.4 *Pre-peak deformation modulus (Average) vs. height-to-diameter ratio*

The following may be observed from the above figure:-

- The pre-peak deformation moduli for the four standard size specimens vary between 66 MPa and 137 MPa. The average value for the deformation modulus is 106 MPa and the range is 71 MPa ($\approx 67\%$). Again, this variation is large, but may be expected considering the variation in strength.
- The deformation moduli increase with increasing height-to-diameter ratios. This trend is unexpected but may be as result of the reduction of cross-sectional area as mentioned above.

Post-peak deformation modulus

In the post-peak regions of the stress-strain curves the deformation moduli have again been evaluated as ‘average’ slopes. Figure 3.5 shows the ‘average’ post-peak deformation moduli plotted against height-to-diameter ratio.

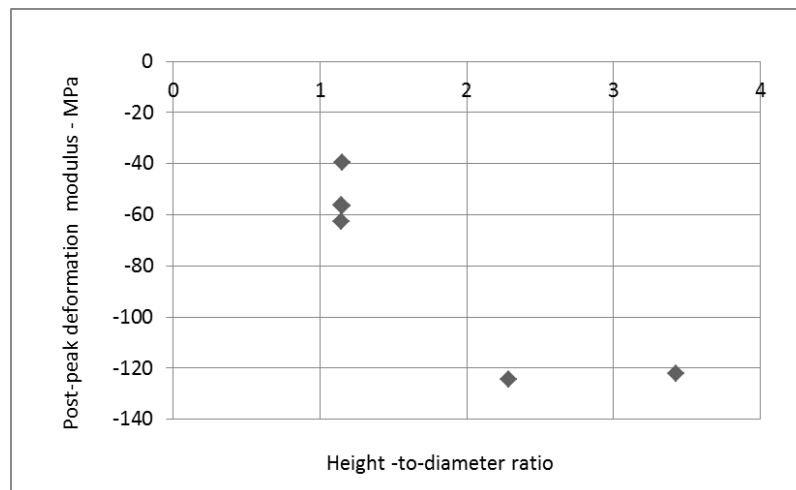


Figure 3.5 *Post-peak deformation modulus vs. height-to-diameter ratio*

The following may be observed from the above figure:-

- Because of the Class I type of behaviour, the values for the post-peak deformation moduli are negative.
- The post-peak deformation moduli for the four standard size specimens vary between -40 MPa and -63 MPa, the average post-peak deformation modulus is -54 MPa and the range is 23 MPa ($\approx 43\%$).
- The values for the post-peak deformation moduli for the two specimens with larger height-to-diameter ratios are significantly lower than the moduli for the four 'standard' size specimens. In this context 'lower' means steeper slopes of the stress-strain curves resulting in larger negative values. There is thus a definite trend of decrease of post-peak deformation modulus with increasing height-to-diameter ratios. The trend has not been defined mathematically with the current results of only six tests.
- The abovementioned observations are significant and may be explained by means of the schematic diagram depicted in Figure 3.6. It must be stressed that this model is real and practical and in line with observations made during this investigation. For purposes of the explanation, straight line trends have been illustrated. In reality the trends are non-linear, but the principles remain the same. In addition, the effect of time (relaxation; creep) has been ignored.

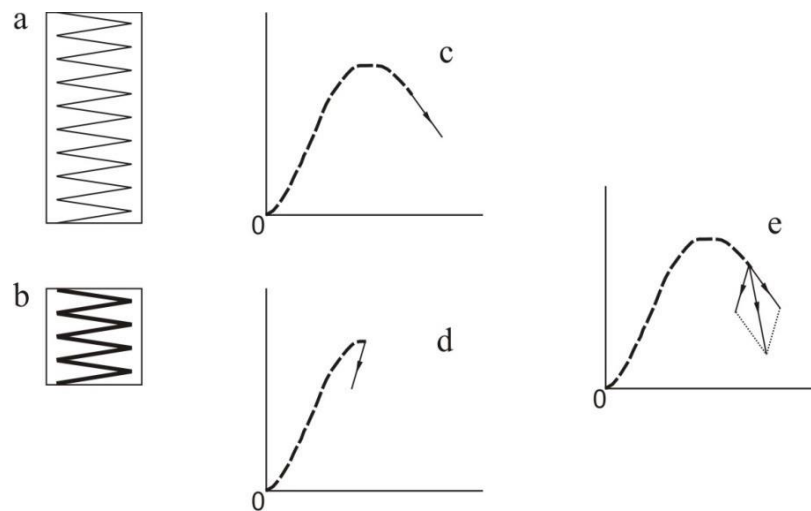


Figure 3.6 *Influence of the ‘less fractured’ part of a test specimen on the post-peak deformation behaviour of the whole*

- a. Shows part (say top part) of the specimen with fractured portions in series with ...
- b. part (say bottom part) of the specimen which is unfractured or less fractured.
- c. The top part of the specimen will deform in the standard fashion as depicted in c. while ...
- d. on reduction of stress the bottom part will react elastically in a much stiffer mode. This is illustrated in d. The elastic components of the deformations may be likened to springs with different stiffnesses.
- e. For the total (combined) specimen the actual slope of the post-peak curve will be a combination of the modes of deformation of the parts.

From the above explanation it may be concluded that the longer the test specimen, the more the angle of the post-peak curve will rotate in a clockwise direction. In the case of Class I type behaviour this will result in steeper slopes resulting in higher negative values for the moduli calculated

Elastic modulus on unloading

The shape of the unloading curve of the second, post-peak unloading cycles is continuously nonlinear and concave (see Appendix A). The ‘Tangent’ slopes of the unloading parts of the stress-strain curves have thus all been determined at a stress value of 0.25 MPa. Only four such unloading curves were available for evaluation. ‘Secant’ slopes would have been impractical because the stresses at the beginning and the end of the unloading curves were not the same for all the tests. Figure 3.7 below shows the tangent elastic moduli plotted against height-to-diameter ratio.

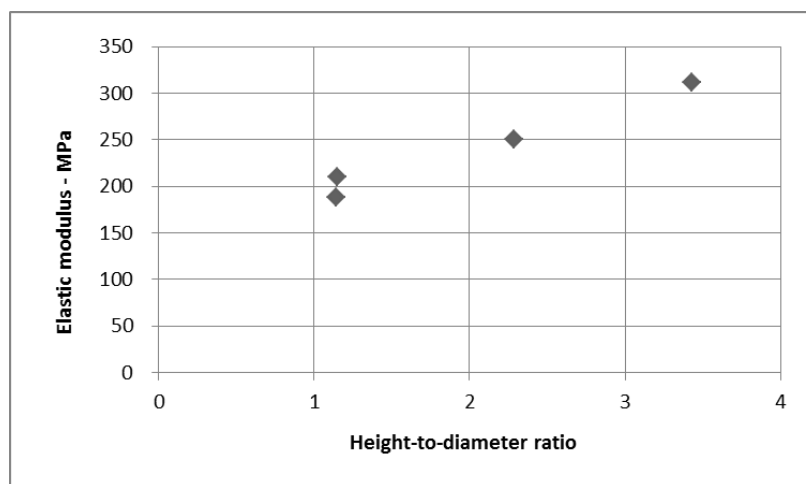


Figure 3.7 *Elastic modulus (on post-peak unloading) vs. height-to-diameter ratio.*

The following may be observed from the above figure:

- There were only four tests with unloading and re-loading cycles.
- There is a significant direct relationship i.e. increasing E-modulus with increasing height-to-diameter ratio.
- Increasing E-modulus means increasing stiffness which in turn means less negative strain increment for the same stress decrease.

The abovementioned observations are very significant and may be explained by means of the schematic diagram depicted in Figure 3.8.

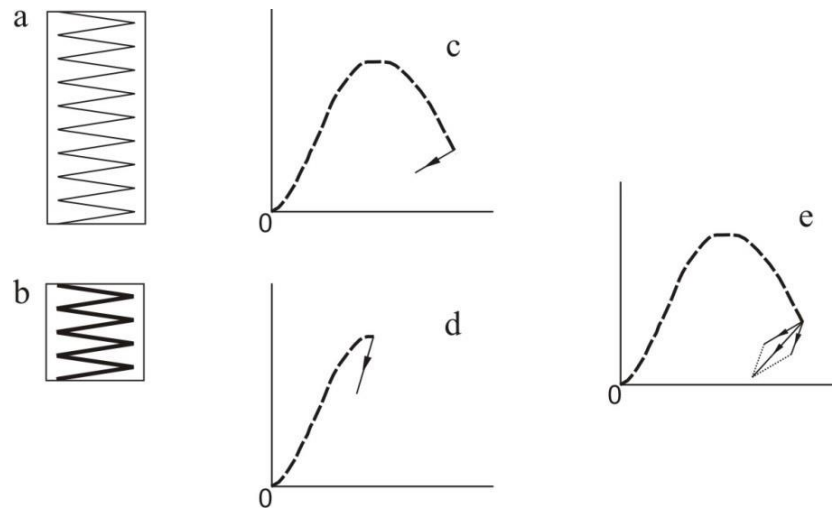


Figure 3.8 *Influence of the ‘less fractured’ part of a test specimen on the elastic deformation behaviour on unloading of the whole*

- a. Shows part (say top part) of the specimen with fractured portions in series with ...
- b. part (say bottom part) of the specimen which is unfractured or less fractured.
- c. During the unloading cycle in the post-peak region of the stress-strain curve the top part of the specimen will deform in the standard fashion as depicted in c. while
- d. On reduction of stress the bottom part will react elastically in a much stiffer mode. This is illustrated in d. The elastic components of the deformations may be likened to springs with different stiffnesses.
- e. For the total (combined) specimen the actual slope of the unloading curve will be a combination of the modes of deformation of the parts.

From the above explanation it may be concluded that the longer the test specimen, the more the angle of the tangent to the unloading curve will rotate in an anti-clockwise direction. This rotation will result in steeper slopes resulting in higher positive values for the elastic moduli.

There is thus a significant direct relationship i.e. increasing E-modulus, on final, post-peak unloading, with increasing height-to-diameter ratio.

Angle of predominant failure plane

In Appendix A, a number of photographs are reproduced together with the documentation of additional information.

The following may be observed from the photographs:

- For the ‘standard’ shape specimens with height-to-diameter ratio of 1.1 the dominant failure plane is roughly diagonal. The angle between the plane and the axis of the cylindrical specimen is therefore approximately 45°. For a typical example see Figure A8.
- Similarly, for the specimen with height-to-diameter ratio of 2.3, the main failure plane is again roughly diagonal. The angle between the plane and the axis of the cylindrical specimen is now approximately 24°. See Figure A10.
- Contrary to the above two observations, for the specimen with height-to-diameter ratio of 3.4, the main failure plane is **not** roughly diagonal. For this specimen the angle is approximately 21°. See Figure A11.
- It may thus be seen that for specimens with greater heights a relatively larger portion of the total specimen is less fractured than the rest.

A plot showing the angle of the predominant failure plane vs. test specimen height-to-diameter ratio is given in Figure 3.9. For the shorter specimens the angle of failure is equal to the angle of the diagonal of the specimen. For the longest specimen the angle is not equal to the angle of the diagonal but might have levelled off at the height-to-diameter ratio of about 2.5 as is the case for the Anorthosite which will be reported in a later section.

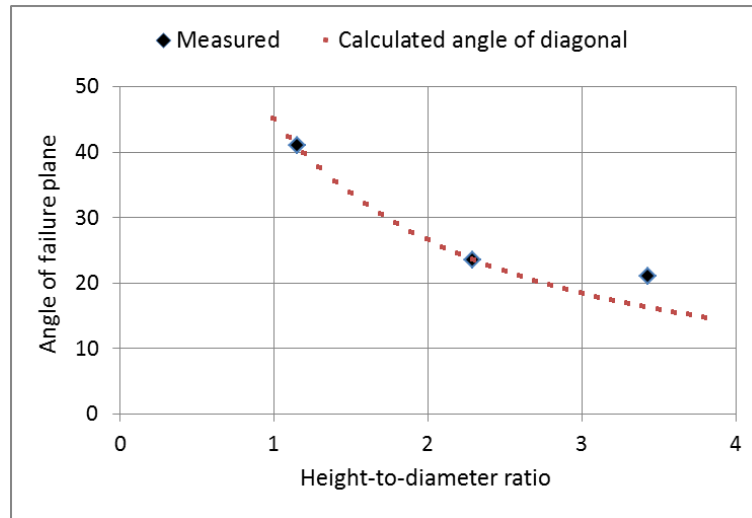


Figure 3.9 *Angle of predominant failure plane vs. height-to-diameter ratio*

3.2.6 Conclusions of tests on stabilized soil

- The stabilized soil has very typical Class I type behaviour.
- The influence of the test specimen geometry was investigated by varying the height-to-diameter ratios of the test specimens. It was confirmed that the stress-strain behaviour of the stabilized soil material in the post-peak region is influenced significantly by the geometry of the test specimens.
- The behaviour is thus not only a ‘material characteristic’, but also a ‘specimen characteristic’.
- The findings emphasize the importance of standardized test methods to allow assessment and comparison of results of tests.
- This type of artificial model material offers possibilities of further studying the post-peak behaviour of Class I type materials by using ‘soft’ testing machines and simple, inexpensive equipment.

3.3 Determination of the post-peak behaviour of spotted Anorthosite

3.3.1 Motivation for selecting Anorthosite

For testing the premises of this dissertation, a rock material capable of delivering repeatable test results was necessary. The requirements for such a rock material may be summarized as follows:

- The rock material had to be reasonably homogeneous so as to provide repeatable results. In published Rock Mechanics literature there are many precedents for using igneous rocks for research tests. For example Bieniawski (1967a) and Ozan and Canbulat (1994) used Norite, Westerly Granite was used extensively in the USA by Wawersik and Brace (1971).
- The rock should preferably be ‘medium strength’ to ‘high strength’ i.e. the strength should be between 50 MPa and 200 MPa according to the classification by Deere and Miller (1966). By selecting a rock material from the stated strength range it was hoped that it would exhibit Class II behaviour and will at the same time not be too brittle so as to allow the available servo-controlled testing equipment to control and trace the complete stress-strain curves.
- The rock should have a reasonable Poisson’s ratio (ν), i.e. during loading it should exhibit sufficient radial expansion for the circumferential transducer to measure and to serve as input signal for the servo-controller of the testing machine. In the authors opinion ν should preferably be > 0.2 .

Anorthosite was considered for this series of tests because it was hoped that it will meet with the abovementioned requirements. In addition, Anorthosite is relatively easy to obtain since it is present in the Bushveld Igneous Complex. It is abundant in the mines around Rustenburg / Marikana / Brits / Northam where it is found above as well as below the Merensky Reef. It is thus a very relevant rock type with regard to platinum mining taking place in the Bushveld Complex.

3.3.2 Preliminary test to confirm the suitability of Anorthosite

A preliminary test was performed on an Anorthosite specimen to ensure that this would be a suitable rock type for the main testing programme. Details of the test are presented in Appendix B. The observations made and the conclusions reached are briefly summarized below:

- The strength of the Anorthosite specimen tested is 97 MPa. This strength falls in the upper region of a ‘medium’ strong rock. It was thus expected to be a rock exhibiting Class II type behaviour.
- Load cycling does not significantly influence the subsequent stress-strain behaviour.
- The Anorthosite shows high lateral strain which is expected to result in sufficiently large signal strength for feedback control for the planned research tests. The tangent Poisson’s Ratio calculated at 50% of the strength even exceeded 0.5, a value that is exceptionally high.
- It could be expected that the stress-strain behaviour can be successfully monitored in the post-failure region when tested in a servo-controlled testing machine.
- The Anorthosite was thus accepted for the research tests intended for testing the premise of this dissertation.

3.3.3 Selection of variables for testing the Anorthosite

The aim of the series of research tests was to determine the influence of the geometry of test specimens on their behaviour. In effect this meant that test specimens with different height-to-diameter ratios had to be tested. The decision was made to perform triaxial compression tests to improve the chances of successfully monitoring the complete stress-strain curves even if specimens with non-standard, extra-large height-to-diameter ratios are to be tested.

A relatively low confining pressure of 10 MPa was selected and was kept constant during all the tests. The confining pressure was considered ‘relatively low’ because it was only about 10 % of the UCS of the Anorthosite as determined during the preliminary test (UCS = 97 MPa, see Appendix B). Using this confining pressure was expected to improve the chances of successfully obtaining complete post-peak curves.

The consensus in the published literature is that the elastic energy stored in the test specimen is responsible for the fact that many types of rocks exhibit Class II type behaviour. To investigate this it was considered absolutely necessary that load cycling, i.e. unloading and subsequent reloading, be performed.

3.3.4 Test equipment

The tests were performed in the MTS servo-controlled testing machine available in the Genmin Laboratory of the University of the Witwatersrand (Wits). As mentioned above, triaxial tests were performed. Photographs of the equipment are shown in Figures 3.10 and 3.11.



Figure 3.10 *The MTS servo-controlled testing machine*



Figure 3.11 *The MTS Transducers for axial and circumferential strain determination*

Notes: The axial transducer always measures over a constant length of 50 mm.

Only one axial transducer was available (the ISRM SM suggests 2).

The circumferential transducer measures the average radial strains.

The physical dimensions of the available triaxial cell dictated the dimensions of the test specimens. The maximum length of test specimen plus loading platens (end caps) recommended by the manufacturers is 214 mm. Core diameters of 42 mm were chosen and thus the maximum length-to-diameter ratio that could eventually be tested was 3.5, with the nominal range from 1.5 to 3.5. Extra-long and extra-short loading platens had to be manufactured so that the height of test specimen plus applicable loading platens was suitable for the test cell. Figure 3.12 shows 3 specimens with different height-to-diameter ratios between loading platens with different heights.



Figure 3.12 Test specimens with different height-to-diameter ratios

Note: The photograph was taken after testing.

3.3.5 Test procedure

The test procedure adopted is described in detail in the next paragraphs using the axial stress vs. time graph for test specimen number An-05 as example. The programme for each test consisted of several phases as illustrated in Figure 3.13 below showing the axial stress vs. time curve for specimen An-05.

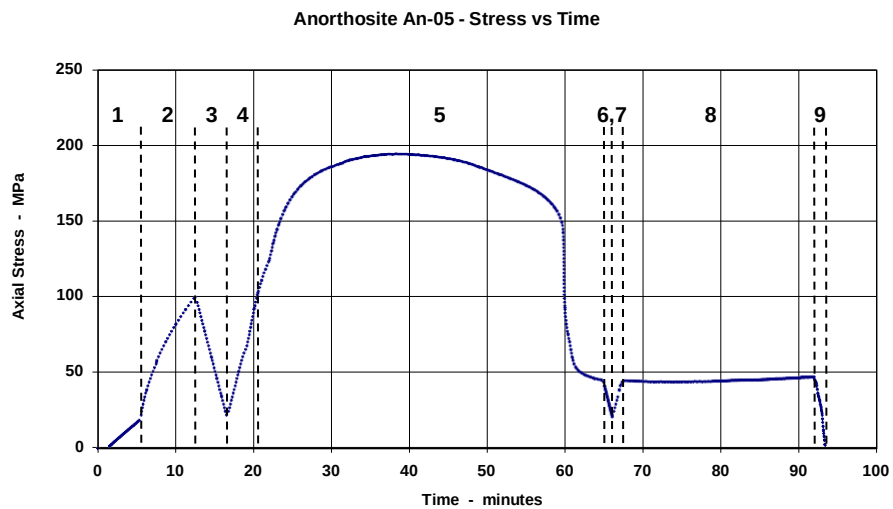


Figure 3.13 Axial stress vs. time for test specimen An-05

The test phases were:

Phase 1: Initial phase. After a minor 'zero' load, the confining hydraulic pressure was applied simultaneously with the axial stress. Ideally, as suggested in the ISRM SM (ISRM, 2007b) the axial stress should be applied in 'hydrostatic' mode i.e. the axial stress σ_{ax} should be equal to the confining stress i.e. the hydraulic pressure P . In practical reality it is always advisable to ensure that $\sigma_{ax} > P$ to avoid the loading piston from losing contact with the specimen. This phase lasted until the hydraulic pressure of 10 MPa was reached.

Phase 2: Application of axial load. The constant confining pressure of 10 MPa was maintained while the axial load was gradually increased. The rate of load application was automatically controlled by the servo controller in such a way that a circumferential strain rate of approximately 0.36 millistrain per minute was maintained. One millistrain unit is equivalent to 0.001 m/m or 0.1%.

Phase 3: First unloading cycle. This unloading cycle automatically commenced at a stress level of 100 MPa. Unloading took place at a rate of 25 kN per minute, corresponding to a stress rate of about 18 MPa per minute. Thus unloading from 100 MPa to 20 MPa took approximately 4.5 minutes.

Phase 4: Reloading at the same stress rate of 18 MPa per minute to approximately 100 MPa.

Phase 5: At this stress level of approximately 100 MPa further load increase was again automatically regulated by the servo controller at the abovementioned circumferential strain rate of approximately 0.36 millistrain per minute. This was continued until and beyond the peak stress (strength).

Phase 6: Second unloading cycle, controlled as during phase 3 unloading. The timing of the second unloading cycle changed during the testing programme as will be discussed in Section 4.3 of the dissertation.

Phase 7: Second reloading cycle, again as during phase 4.

Phase 8: Final loading phase, again regulated at approximately 0.36 circumferential millistrain per minute. This phase was continued until the curves were levelling off.

Phase 9: Eventual unloading.

3.3.6 Methods of evaluation of test data

Evaluation of deformation characteristics

All the test data was available in the form of Excel files. The number of data sets varied between 580 and 1250, depending on the number of cycles performed. Graphs showing the curves were printed out. The relevant parts of the curves were then selected and enlargements of these sections were again printed out. Best fit tangent lines were drawn by eye to these curves and two sets of co-ordinates on the tangent lines were selected for calculation of the values. Where the data points were too close together for accurate calculations, a parallel line was drawn intersecting two suitable points.

As was expected, all the specimens exhibited Class II type behaviour. In the case of the slopes in the post-failure regions the least steep parts of the curves were selected. This least steep slope is the region where the influence of the elastic rebound is at its maximum.

Evaluation of work / energy recovered

As has been stated before, the work done or the energy used is equal to the area under the load-deformation curves. If the units for load are Newtons (N) and the displacements or deformations are expressed in meters (m) then the units of the abovementioned areas are Joules (J). Similarly, the area under the stress-strain curve is equal to the work done or energy used per unit volume or J/m^3 .

The elastic energy released by the specimen during class II behaviour is illustrated by means of Figure 3.14 which shows the elastic energy released per unit volume of test Specimen An-05 in the post failure region of the stress-strain curve. The horizontally shaded area represents the energy released. Part of this energy i.e. the vertically shaded area, is subsequently required for further deformation. The method of evaluation the energy has been described in detail in section 2.3.1.

In addition, the total work done to deform the specimens from zero to a strain of 0.7 % or 7 mε was calculated. Testing had been done until the stress-strain curves were levelling off and in two cases the tests were terminated prior to reaching the 0.7 % strain. For purposes of evaluation the latter curves were extrapolated slightly, which was acceptable because the curves had levelled off anyway.

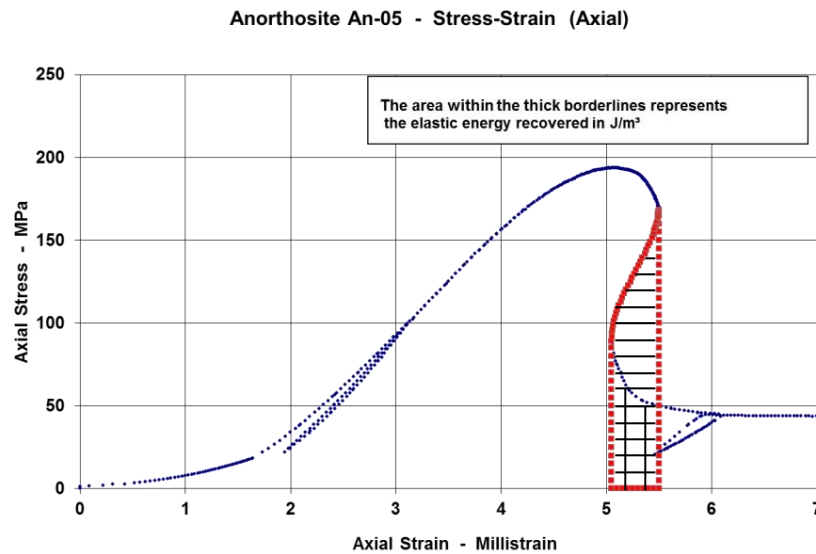


Figure 3.14 *Elastic energy released during Class II behaviour*

Determination of the angle of failure

In the literature it has been described that failure is “*highly localized*” (Ryder and Jaeger, 2002). Such ‘localized’ failure planes were indeed experienced for all the successful tests described in this chapter. Actually, all the failure planes were single planes and were roughly flat, particularly for the test specimens with height-to-diameter ratios > 2. Figure 3.15 below shows examples of single failure planes. Individual photographs for all the specimens are documented in Appendix C.



Figure 3.15 Mode of failure of Anorthosite test specimens

Three methods for measuring the angles of the planes relative to the axis of the test specimens were attempted.

- The specimens were photographed in front of a graduated scale and the angles were estimated. See the photographs in Appendix C.
- The angles were measured with a protractor.
- The third method was to measure the distances between the positions where the failure planes intersected the surfaces of the specimens. A simple trigonometric calculation then provided the angles.

The values obtained by the three methods were checked and re-checked until they agreed to within 2°.

The two specimens with height-to-diameter ratios of 1.56 and 1.96 showed minor signs of development of cones. The diagonal was however the best estimate for the angle of failure. For the rest of the specimens the planes of failure were almost flat.

Determination of the ultimate resistance to sliding

The final loading phase before unloading resulted in stress-strain curves which were practically level indicating a constant resistance to sliding. The information gained from this behaviour is often used to characterize the frictional behaviour of fractured rocks (Drescher, 2010).

Beyond the strain of about 0.7 % most of the stress-strain curves were levelling off. The ultimate values (in MPa) were measured and the results have been added to the tables of results.

3.3.7 Presentation of test results

The results of the tests are summarized in Table 3.2.

Graphs for the individual test specimens have been reproduced in Appendix C. Each page in the appendix presents the following information for each specimen.

- Circumferential strain vs. time
- Stress (Axial) vs. time
- Stress (Axial) vs. strain
- A photograph of the specimen after failure against a graduated background
- A text box with some selected results
- A second text box with additional, test specimen specific notes.

Table 3.2 Results of post-peak testing of spotted Anorthosite

TEST PARTICULARS			SPECIMEN DIMENSIONS						RESULTS OF TESTS						WORK / ENERGY				Notes
Test No	Test Specimen No	Nominal Height to Dia Ratio	Dia. (mm)	Height (mm)	Actual Height to Dia Ratio	Mass (g)	Volume m³	Density (kg/m³)	Strength (MPa)	Tan @ 50% Strength		Post Peak Modulus 'M'	Angle of Failure (°)	Final sliding Resistance (MPa)	Post Peak Energy recovered		Total Area under curve from 0 to 7mε		
										Def. Modulus (GPa)	Modulus Ratio				J	kJ/m³	J	kJ/m³	
An-04	TCP-02	2.0	42.0	79.8	1.90	318.1	0.000111	2870	188.0	n.a.	n.a	n.a.	28.0	n.a.	n.a.	n.a.	n.a.	a, b	
An-05	TCP-D	2.5	42.0	103.3	2.46	414.0	0.000143	2890	193.9	64	330	129	24.0	45.0	8.4	58.8	74.8	522.6	c d
An-06	TCP-C	3.5	42.0	146.8	3.49	579.0	0.000203	2850	188.9	64	339	126	23.7	48.2	11.7	57.7	97.9	481.2	
An-07	TCP-E	2.5	42.0	105.0	2.50	418.5	0.000145	2880	189.5	60	317	n.a.	26.0	n.a.	n.a.	n.a.	n.a.		
An-09	TCP-03	2.0	42.0	82.2	1.96	328.2	0.000114	2880	208.2	65	312	194	27.0	61.1	n.a.	n.a.	78.5	689.5	
An-10	TCP-04	2.5	42.0	103.9	2.47	411.2	0.000144	2860	193.4	58	300	115	24.6	52.0	11.3	78.3	90.6	629.4	
An-11	TCP-01	1.5	42.0	65.6	1.56	262.1	0.000091	2880	222.3	67	301	198	32.6	84.2	3.2	35.4	61.0	670.4	
An-12	TCP-05	3.0	42.1	125.3	2.98	506.0	0.000174	2910	188.2	54	288	113	24.1	52.8	9.8	56.3	111.6	641.0	
An-13	TCP-A2	2.5	42.0	105.9	2.52	426.0	0.000147	2900	189.7	56	295	133	24.8	50.0	6.0	41.1	89.8	612.2	
An-14	TCP-F	2.5	42.0	103.9	2.47	416.0	0.000144	2890	183.9	67	364	129	25.3	44.2	10.9	75.6	77.3	536.7	
An-15	TCP-06	3.5	42.0	143.6	3.42	572.9	0.000199	2880	187.1	54	287	91	23.9	40.0	22.7	114.0	103.9	523.0	

Notes:- a. All data lost due to power failure
b. Strength of 188 MPa is low estimate. The value was obtained from notes made sometime before event
c. Post peak data lost due to loss of control on change over in control mode
d. Energy calculations not possible due to excessive influence of load cycling

For the description of a typical result the stress-strain-curve for test specimen An- 05 is used as example. The axial stress-strain-curve for the specimen is shown in Figure 3.16 below. The dimensions of the specimen, including the height-to-diameter ratio, are listed in the text box.

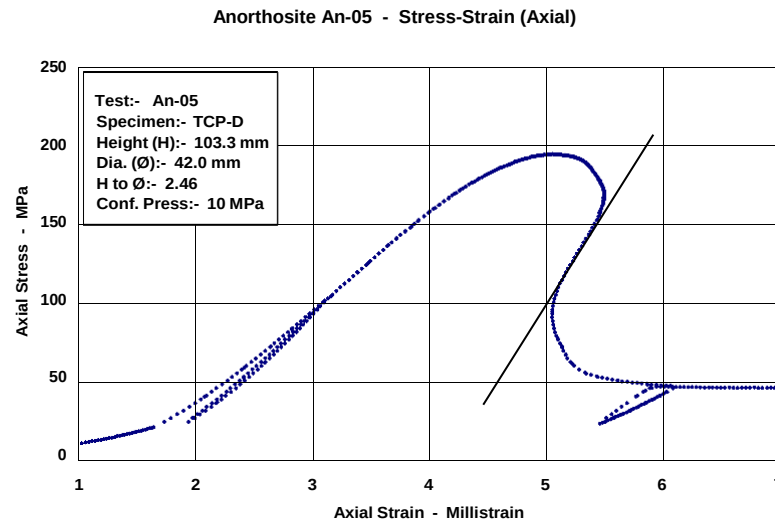


Figure 3.16 Stress-strain curve for test specimen An-05

The Class II behaviour of the specimen with height-to-diameter ratio of 2.46 is obvious. The minimum slope of the post-peak curve (M) was found to be 129 GPa. At peak strength the stress is near constant for a while although axial strain increases while the circumferential strain rate is regulated automatically at the pre-set rate of 0.36 millistrain per minute. The failed specimen behaves elastically during the post peak unloading followed by reloading cycle.

Individual data points have been plotted and were not connected. Test data is sampled at a fixed, pre-programmed rate, thus the distance between successive points is an indication of the rate of change which, in the above case, is the change in axial stress and axial strain. The number of data points and the distances between points clearly show that the testing machine was in perfect control throughout the duration of the test. Sudden, uncontrolled failure would have resulted in big distances between points.

3.3.8 Discussion of the results of tests on Anorthosite

The effect of height-to-diameter ratio on strength

As was expected (John, 1972) the compressive strength was higher for the short specimens but was practically constant for specimens with height-to-diameter ratios ≥ 2.5 . Although John performed uniaxial tests, the results of the current series of tri-axial tests show the identical trend. This is illustrated in Figure 3.17 below. Please note that the result marked with a circle is a low estimate, because the specimen failed prematurely as result of a power failure causing loss of control. The five results for the strength of specimens with the ‘standard’ height-to-diameter ratio show only minor variations illustrating the uniformity of the Anorthosite selected for the series of tests. The curve levels off at the strength of 189 MPa.

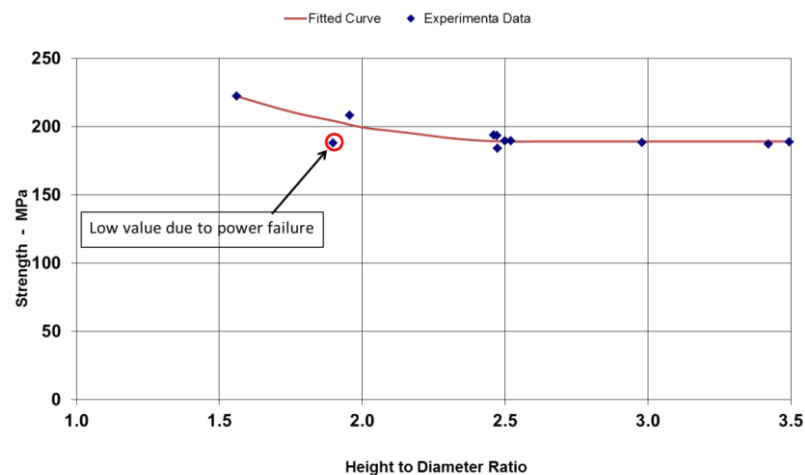


Figure 3.17 *Strength vs. height-to-diameter ratio at confining pressure of 10 MPa*

The effect of height-to-diameter ratio on the angle of failure

A graph showing the angles of the failure planes is presented in Figure 3.18 below. Added to the figure is a line showing the calculated angle of diagonal planes through the specimens.

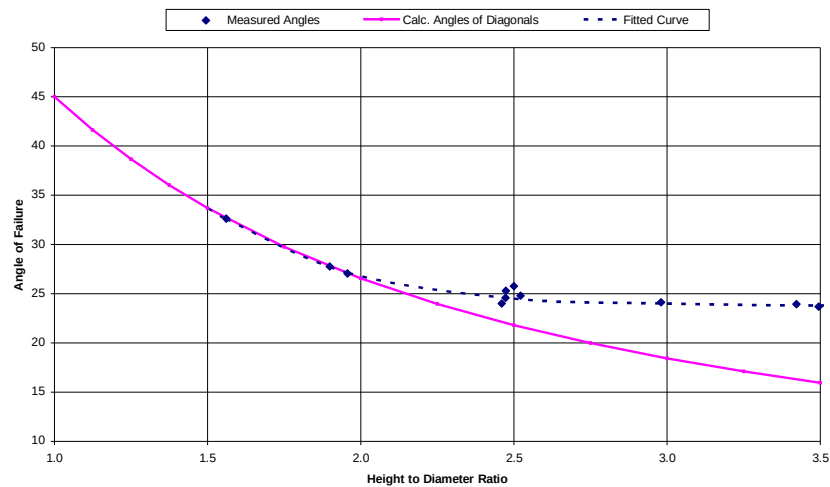


Figure 3.18 Variation of the angle of failure with test specimen height-to-diameter ratio

From the figure it is may be seen that:

- The values for the angles of failure result in an exceptionally smooth curve i.e. there are only very small variations in the results.
- The curve for angles of failure appears very similar to the curve for the strength in Figure 3.18.
- The curve levels off for specimens with height-to-diameter ratios ≥ 2.5 .
- For test specimens with height-to-diameter ratios < 2.0 , the angle of failure is equal to that of the diagonal plane through the specimen.
- The curve levels off at an average angle of 24.3° .

The effect of height-to-diameter ratio on pre-peak deformation behaviour

The lower curve in Figure 3.19 below shows the values for the ‘tangent 50% UCS’ deformation moduli (E) in the pre-peak regions of the stress-strain curves. As was to be expected, the deformation moduli in the pre-peak region of the stress-strain curves are almost constant with an average value of about 60 GPa. Only the two specimens with height-to-diameter ratios < 2.5 appear to have a slightly higher value for the deformation modulus.

An explanation for this is that the relative influence of the constraints exerted by the loading platens is much larger for short specimens than for long specimens and that thus the stress distribution throughout the test specimen is less uniform. It must be noted that the gauge length of the axial transducer is a constant 50 mm for all the tests.

The results confirm that the values of the pre-peak deformation moduli are not affected by the geometry of the tested specimens i.e. their height-to-diameter ratio, provided that test specimens with ‘standard’ height to diameter ratios ≥ 2.5 are tested.

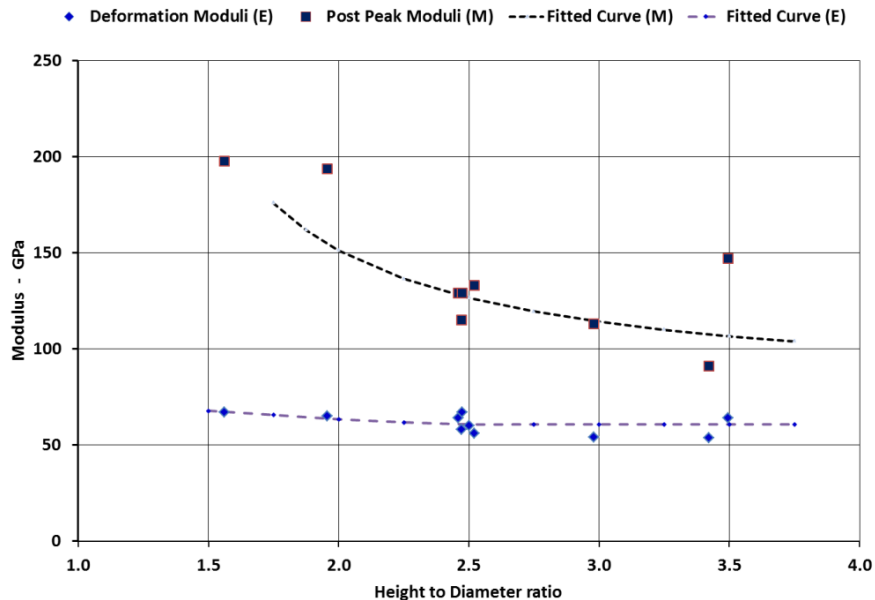


Figure 3.19 Influence of height-to-diameter ratio on pre-peak and post-peak deformation behaviour

The effect of height-to-diameter ratio on post-peak deformation behaviour

The values for the post-peak moduli (M) have been added to Figure 3.19 above. Because all the specimens behaved in the typical class II manner, all the calculations returned positive values. Contrary to the pre-peak deformation behaviour, the post-peak deformation behaviour is significantly influenced by the height-to-diameter ratio of the test specimens and hence by the geometry of the specimens.

This result is regarded as the most important finding of the research for this dissertation. It illustrates the fact that the behaviour of rock in the post-peak region is significantly influenced by the height-to-diameter ratio of the test specimens. The implication is that the deformation behaviour of a partially failed rock material is not only dependent on the properties of the rock material but also on the geometry of the test specimen. The explanation for this behaviour is the effect of the elastic properties of the fragments of the failed specimen. The stresses in such fragments are reduced progressively as the resistance to load of the test specimen diminishes. The stored elastic, strain energy is then gradually released. The released elastic energy depends on the size of the fragments relative to the size of the whole specimen and thus on the geometry of the specimen tested.

Work done or energy required to deform the test specimens

As mentioned in Section 3.3.6 above, the total quantity of work done to deform the specimens to a strain of 0.7 % was calculated. The results are listed in the tables of results (Table 3.3) together with all the other data collected during the evaluation of the test results. Figure 3.20 shows the total work done to deform the specimen in Joules (J). It may be seen that there is a distinct relationship between the work done per specimen and the increasing height-to-diameter ratio. The correlation coefficient (R^2) is a remarkable 0.72 when a straight line is fitted.

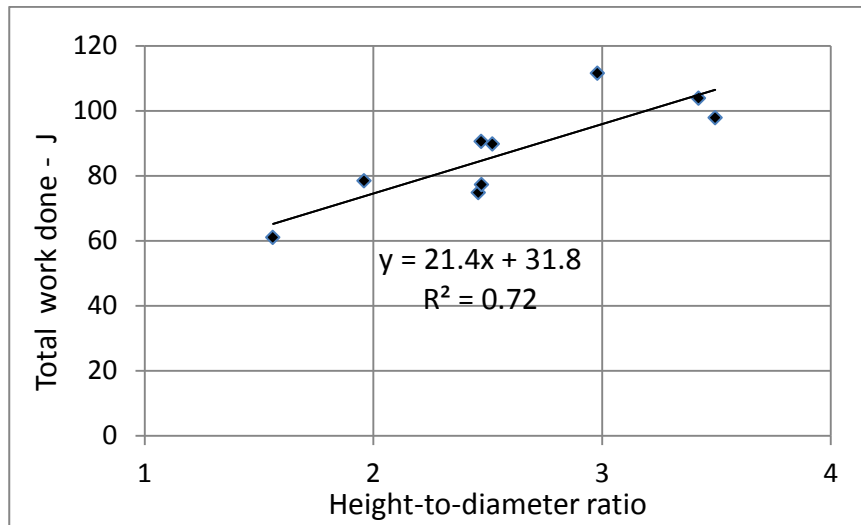


Figure 3.20 Total work done (J) to deform specimens from 0 to 0.7 % strain

The observed trend is to be expected because with increasing height-to-diameter ratio the volume of rock to be deformed to a strain of 0.7 % increases.

Figure 3.21 shows the work done per unit volume (kJ/m³) required to deform the specimens to a strain of 0.7 %. The volumes of the test specimens were small fractions of cubic meters and thus large values for the amount of energy required per cubic meter have to be expected. Unfortunately the results show large scatter. A linear regression resulted in a correlation coefficient R^2 of 0.52.

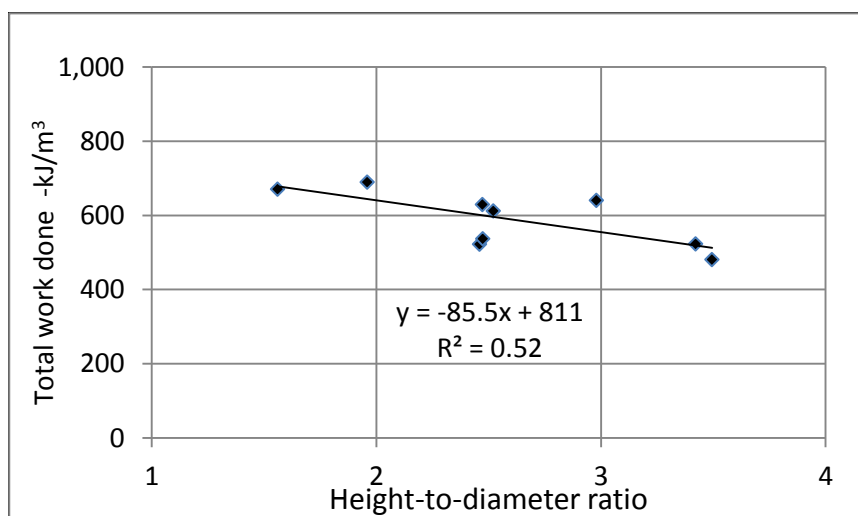


Figure 3.21 Total work done per m³ (J/m³) to deform specimens from 0 to 0.7 % strain

When compared with the results of the energy per specimen, the smaller correlation coefficient may be explained by the introduction of an additional variable for the evaluations i.e. the volume of the test specimens. A definite trend of less energy per unit volume for specimens with larger height-to-diameter ratios is obvious. The data does not allow extrapolation and thus it cannot be stated with certainty whether the trend is linear or not. However, it may be presumed that the trend should be that the curve eventually levels off for specimens with large height-to- diameter ratios.

Energy released by the test specimens in the post-failure region of the stress-strain curves

In the case of materials exhibiting Class II behaviour there is, by definition, always a region of the stress-strain curve where the test specimen is effectively expanding while at the same time losing strength. This phenomenon results in a double negative or, in effect, a positive post-peak slope of the curve.

The excess elastic energy in the test specimen has been quantified by calculating the areas under the curves as was described in Section 3.3.6. The results are presented in Table 3.3 and are illustrated graphically in Figures 3.22 and 3.23 below.

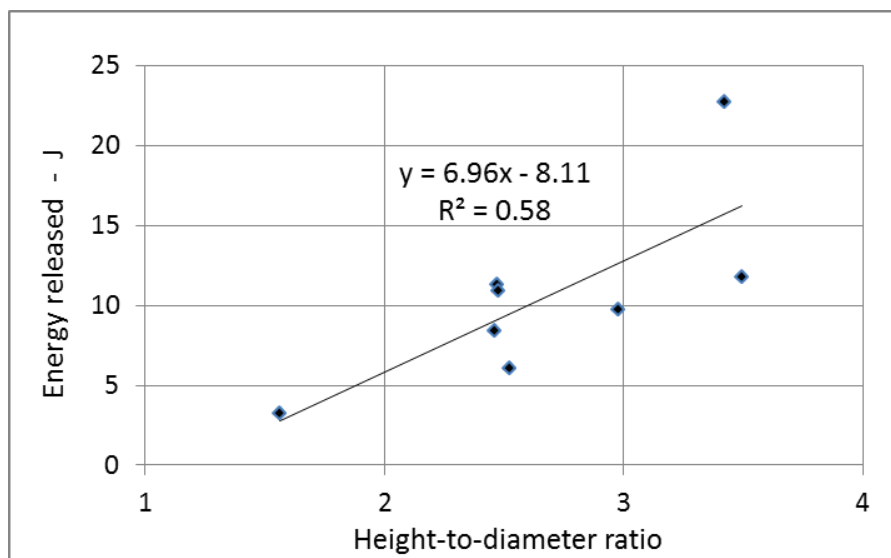


Figure 3.22 Elastic Energy (J) released by specimens in the post-failure region

Figure 3.22 presents the energy released in Joules. It is obvious that there is a direct relationship between height-to-diameter ratio and the energy released. This trend was to be expected when the Class II behaviour is ascribed to the elastic energy stored by the specimen.

It is interesting to note that, for a specimen with height-to-diameter ratio of approximately 1 the graph meets the X-axis and thus there will be no energy released. For this material and for the testing conditions applied it appears as if the behaviour of test specimens with height-to-diameter ratios < 1 will be Class I.

Figure 3.23 illustrates that the energy released per unit volume also exhibits a direct relationship. Unfortunately there is a very large scatter in the values obtained, R^2 being only 0.32.

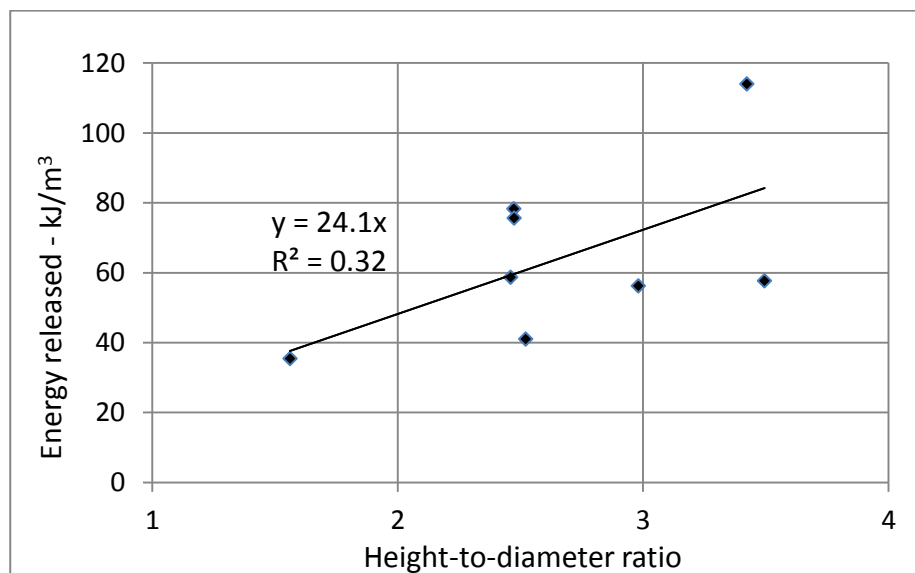


Figure 3.23 *Elastic energy per m³ (kJ/m³) released by specimen in the post-failure region*

All the results obtained during the study of work done and of elastic energy released confirm once again that the behaviour is geometry dependent.

Ultimate resistance to sliding

Figure 3.24 shows the graph of the ultimate resistance to sliding vs. height-to-diameter ratio. The ultimate resistance to sliding is the stress required for further deformation to 0.7 % after the stress-strain curves have levelled off. Here too, as is the case for strength and the angle of failure, the influence of geometry diminishes for height-to-diameter ratios > 2.5 .

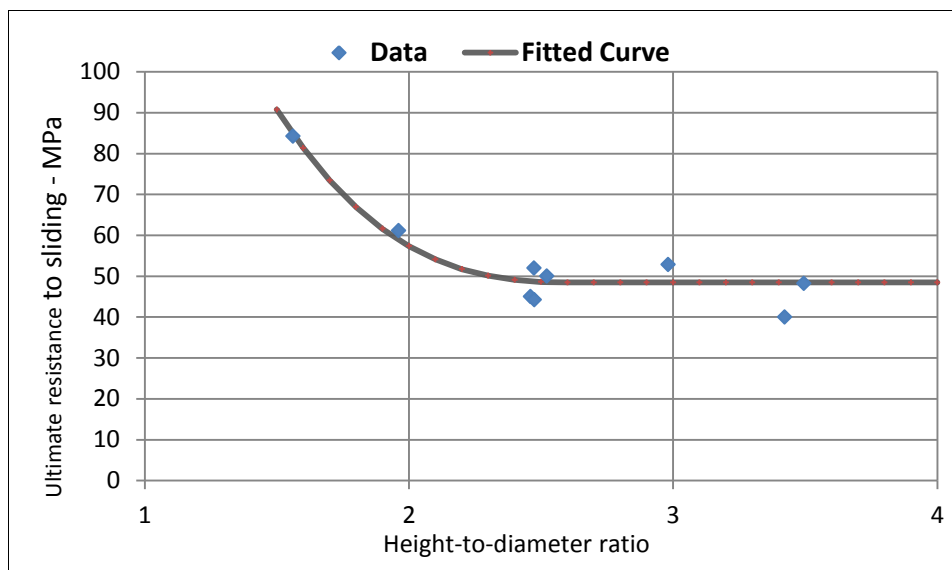


Figure 3.24 *Ultimate resistance to sliding after levelling off vs. height-to-diameter ratio*

The ultimate resistance to sliding is sometimes used to estimate the frictional behaviour of crushed rock (Drescher, 2010). This practice should be used with circumspection because it has been shown above that the results are geometry dependent. The finding is of particular importance because often short specimens are used for testing in order to improve the chances of obtaining the full stress-strain curves.

3.4 Additional notes on the results of the experimental programme

The aim of the experimental programme was to prove the premise that the post-peak behaviour of rock materials is geometry dependent. Two types of materials were tested. For both materials, one with Class I behaviour and the other with Class II behaviour it was possible show that the premise is correct.

The conclusions reached from the experiments described in Chapter 3 will be summarized and discussed in Chapter 5 on conclusions and recommendations. In the next section additional insights resulting from the investigation will be described.

CHAPTER 4

ADDITIONAL INSIGHTS RESULTING FROM THE INVESTIGATION

4.1 Comments on measurement of axial deformation in the post-peak region

The main aim of using stiff, servo-controlled testing equipment is to determine the post-peak behaviour of rock specimens. In the case of the MTS 815 rock testing system the axial deformations can be measured by two independent transducers i.e. the built-in piston displacement transducer or an external transducer clamped onto the surface of the test specimen.

4.1.1 Piston displacement transducer

A piston displacement transducer measures the overall, total movement of the piston of the testing machine. Apart from the deformation of the test specimen, the measurements may include additional displacements such as the deformation of the spherical seat and all the spacers. Should there be any mismatch, foreign particles or minor ridges caused by accidental dents on the interfaces between any of the contact surfaces their deformations will also affect the displacement of the cylinder. ‘Mismatch’ between contact surfaces may include non-flatness and non-parallelity of test specimen ends. In addition, the elastic deformation of the testing machine components, however little, will be added into the piston displacement measurements. Unless calibrations are done and appropriate corrections are made, the additional displacements will in effect be attributed to the test specimen.

4.1.2 The MTS axial transducer

The transducer supplied by the MTS is based on strain-gauged cantilever technology and is clamped onto the side of the rock test specimen (See Figure 3.11 in Section 3.3.4). The main advantage is that it measures the deformation directly on the test specimens. The gauge length of 50mm of the transducer installed at mid-height spans a region of the specimen where the stress distribution and hence the strain distribution may be assumed to be uniform. This comment is obviously only applicable to the pre-peak phase of the tests. In the post-peak phase the specimens are developing: “...*extremely localized failure plane(s)*” (Ryder and Jager, 2002). Because the clamp-on transducer measures the overall average deformation between the points of contact, the resulting measurements are very much dependent on its exact position relative to the developing failure planes. See illustration below.

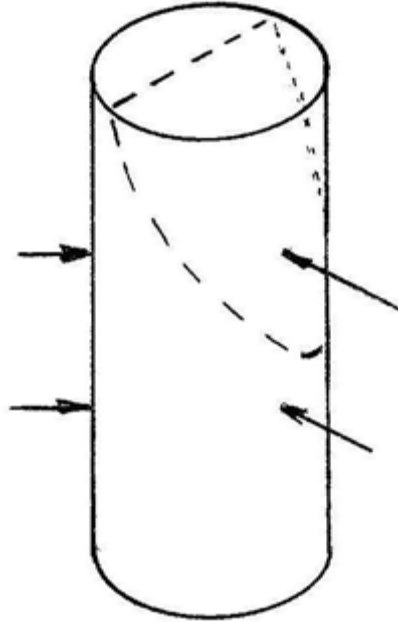


Figure 4.1 *Possible influence of positioning of axial transducers*

4.1.3 Hybrid axial strain measurements (after Drescher, 2010)

Drescher (2010) was aware of the abovementioned advantages and disadvantages and has used a hybrid measuring system when performing tests using the Wits MTS test system. Strains (deformations) were measured by means of both transducers. The secant moduli calculated from measurements between 45 % and 65 % of the UCS during the pre-peak phase by the two transducers were compared and a correction factor was established. This factor was usually between 0.4 and 0.5. On completion of the tests the correction factor was used for evaluating and reporting the axial strains. The method described was used for the evaluation of the results of the tests on Anorthosite conducted for this dissertation.

4.2 Comments on measurement of transverse deformations in the post-peak region

The terms transverse-, lateral-, radial- or circumferential strains are alternative names for the same quantity. If uniform strain conditions in a test specimen are assumed then the values for the quantities are identical. However, depending on the type of transducer used for measuring the quantity either circumferential or radial deformations are measured.

Regardless of increasing stress in the pre-peak phase or decreasing stress in the post-peak phase of a test and regardless of Class I or Class II behaviour of the specimen in the post-peak phase, the transverse strain will always be increasing monotonically. This phenomenon thus makes the output of circumferential transducers ideal as feedback signals for programming and regulating the applied axial load throughout the test. With very few exceptions this is what researchers have used when exploring Class II behaviour by performing tests with servo-hydraulic testing machines. This is also the method recommended in the ISRM SM (ISRM, 2007d).

In published information the transverse strain has received very little attention when reporting results from stiff testing of rocks other than that it was used as input signal for control purposes.

4.2.1 Influence of transverse strains on the effectiveness of the feedback control during servo-controlled testing

Rock materials exhibiting large transverse strains provide large output signals which make measurement and hence control of the tests more accurate. Therefore, the larger the transverse strains, the more effective the control.

The success or otherwise of post-peak testing depends on the stiffness and, even more importantly, on the response time of the servo-hydraulic test system. The response time is of particular importance when specimens with Class II type behaviour are being tested. The formation of new cracks and the extension and opening of existing cracks are time dependent processes. Crack formation and growth will result in circumferential strains and, being time dependent, more crack formation will require more time and thus the testing machine can react relatively faster.

The process of crack formation and crack extension, which contributes to transverse strains, requires energy. Assuming that a particular type of rock with large transverse strains in the pre-peak phase of a test will also have large transverse strains in the post-peak phase, it may be postulated that rocks with higher transverse strains will use up more energy and will behave less brittle.

4.2.2 The use of transverse strain measurements for understanding failure mechanisms of rock

Measurement of axial and transverse strains and subsequent calculation of volumetric strains followed by plotting of the three curves has been found very valuable for understanding of the failure mechanisms of rock in the pre-peak

region (Bieniawski, 1967b), (Li et al, 2011). This was discussed in detail in section 2.1. Stacey has proposed a simple extension strain criterion for fracture of brittle rock which was based on precise measurements of transverse strains. (Stacey, 1985). Transverse strain measurements should therefore be made, interpreted and reported in more detail.

4.2.3 Transverse strain and brittleness of rocks

In very general terms it may be stated that rocks with higher strengths behave in a more brittle manner than rocks with lower strengths. Also, the general trend is that stronger rocks have higher E-moduli. Deere and Miller (1966) have established that most rocks have modulus ratios of between 100:1 and 500:1, indicating a strong relationship between modulus and strength.

A higher modulus (E) means less axial deformation (ϵ_a) for the same stress increment. Transverse strains (ϵ_t) are commonly quantified as Poisson's ratio (ν) which is the ratio between transverse strain and axial strain. If a rock with a higher value for E has the same value for Poisson's ratio it means that the transverse strain is smaller. If such a rock has a small value for Poisson's ratio, then the transverse strain is even less.

It is therefore most likely that strong rocks with small Poisson's ratios behave in a more brittle way than similar rocks with larger Poisson's ratios. The observation has been confirmed by Chen (2012). For purpose of comparison between different materials a 'transverse deformation modulus' (T) has been defined as follows:

$$T = \sigma / \epsilon_t$$

where σ is actual stress and ϵ_t is transverse strain.

T may be calculated from the values for Young's modulus (E) and Poisson's ratio (ν) as follows:

$$T = E / \nu$$

This is because $E = \sigma / \epsilon_a$ and $\nu = \epsilon_l / \epsilon_a$. The unit for T is GPa. Values for T have been calculated for a few selected materials. The results are listed in Table 4.1. For comparison values for diamond and glass, materials which are known to be very brittle, have been added to the table.

Table 4.1 Transverse deformation moduli (T) for selected materials

Material	Source of Information	E GPa	ν	T GPa
Diamond	Internet: WWW.chm.bris.ac.uk/motm/diamond/diamondprop.htm	1220	0.20	6100
Quartzite	Vogler (2012) Peninsula Formation Quartzite	84	0.13	646
Chert	Lama and Vutukuri (1978) USA, Okla. Chalcedony	55	0.16	344
Glass - Soda lime	Internet: WWW.glassarmonica.com/science/elasticity_glass.php	69	0.24	288
Lava	Drescher (2002) Ventersdorp Lava	88	0.31	284
Basalt	Lama and Vutukuri (1978) Howard Prairie Dam	63	0.25	252
Granite	Deere and Miller (1966) Pikes Peak, Colorado Springs	71	0.31	229
Quartzite	Drescher (2002) Witwatersrand, Elsburg formation	47	0.24	196
Marble	Deere and Miller (1966) West Rutland, Vermont	60	0.34	176
Anorthosite	Vogler (2014) Appendix B of this dissertation	39	0.50	78
Coal	Lama and Vutukuri (1978) Poland upper Silecia	5.5	0.44	12.5

From the listing it may be seen that the known brittle materials are high up in the table. For example, the quartzite with a T value of 646 is the same material for which violent failure damaging the circumferential transducer was described in Section 2.4.1. The use of the transverse modulus T for assessing brittleness of rock behaviour appears to be promising and should be further investigated.

4.3 Observations made during load cycling

Cycling has been recommended by the ISRM test method to study the elastic behaviour on unloading. For the first successful test of the series on Anorthosite (AN-05) only one cycle in the pre-peak region of the stress-strain curve and one cycle after levelling off were done. For the next few tests additional cycles were performed. After the next three tests it was realized that unloading cycles during the post-peak phase, i.e. on the descending part of the stress-strain curves, have two detrimental consequences:

- Firstly, the possibility of losing control when changing from strain-control mode to time-control mode. This happened when testing specimen An-07 with the result that no data on the post-peak behaviour could be obtained.
- Secondly, for purposes of this research the slope of the descending part of the curve was of paramount importance. Two of the tests with multiple cycles (tests An-06 and An-09) were successful in the sense that data was successfully captured, but on closer inspection it may be observed that cycling caused problems with the evaluation of post-peak data, particularly also with calculation of work / energy. The values for the post-peak properties for the latter two test specimens are thus probably less accurate than the other values. From test An-10 onwards cycling was therefore

only performed in the pre-peak region of the stress-strain curve and again after levelling off, the same procedure as for Test An-05.

At the time when the tests on Anorthosite (Section 3) were planned, the use of different modes of control for the main tests and for cycling was considered to be the best approach. The programming software was available and operational and proven (Drescher 2002). As mentioned above, it was, however, found that in one case (Test An-07) control was lost on change-over with the loss of valuable information. In retrospect, programming for positive circumferential strain rates for loading and negative circumferential strain rates for unloading might have been a better option.

For future tests the latter control mode is, therefore, recommended.

4.4 Explanation for the phenomenon of localized failure planes

The phenomenon of ‘localized failure planes’ have been reported in literature. (Ryder and Jager, 2002), (Labuz and Biolzi, 1991). The phenomenon has been linked to Class II behaviour. When testing the Anorthosite rock for this dissertation, the observed mode of ultimate failure was indeed that of a single failure plane. (Figure 3.15 of Section 3.3).

An explanation for this observed phenomenon is that, for all successful tests on rocks with Class II behaviour, the excess elastic energy stored in the unfailed parts of the test specimen was indeed effectively removed, leaving only sufficient energy required to continue deforming the specimens at the pre-programmed rate.

Such an explanation has not previously been reported and is therefore an important conclusion reached from the observations made during the current

testing programme. A notable consequence of this finding is that the degree of fragmentation depends on the success or otherwise of rapidly removing any excess elastic energy from the test specimen in addition to removing the elastic energy stored in the equipment. The phenomenon should be further researched for different types of rocks. It is most likely that even when rocks with Class I type behaviour are tested by means of stiff servo-controlled equipment, the predominantly single plane mode of failure will result. However, this should be confirmed experimentally.

4.5 Suggestion for using a hybrid testing system

It has been reported (ISRM, 2007d) that test specimens of extremely brittle rock fail abruptly or even explosively. This may happen even when tests are performed using servo-controlled equipment and when applying the most favourable conditions such as: “*stiff testing frame, slow loading rate, circumferential strain feed-back, low length/diameter ratio*”. This should not happen if the testing equipment is able to release the excess elastic energy timeously.

An explanation for the phenomenon is that elastic energy is released instantaneously on reduction of stress. Servo-controlled testing systems have to measure the control quantity, compare this with the requirements of the test programme, send signals to the hydraulic servo valves which then have to make the necessary adjustments. The latter processes require time and thus the servo-controller may simply not be fast enough.

The use of a hybrid testing system should then be investigated. Such a hybrid system could consist of a combination of stiffening elements, special load and deformation measuring transducers and servo controlled testing machines. A set-

up using stiffening columns has been illustrated and described in Section 2.1 (Figure 2.2).

The insights gained from the current investigation and the explanations for the observed phenomena described in the above chapter have been incorporated in the next Chapter of conclusions and recommendations.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

The experimental work done for this dissertation has shown that the post-peak behaviour of the tested materials is not only a material characteristic but is in addition also dependent on the dimensions of the specimens tested. This conclusion was reached after testing cylindrical specimens with different height-to-diameter ratios of two materials, a low strength material with Class I type behaviour and a medium strength material with Class II type behaviour. The premise to be tested during the research for this dissertation, as was described in Chapter 1, has thus been vindicated. The explanation of the observed behaviour is that in the post-peak phase of a test the elastic energy stored in the unfailed fragments of the test specimen is released while the load bearing capacity diminishes. Release of elastic energy caused by compressive forces results in expansive strain which is in a direction opposite to the compressive strain caused by progressive failure. The relative proportions of the unfailed fragments to the total volume of a test specimen differ for specimens with different height-to-diameter ratios and hence the overall deformation behaviour is affected.

An important consequence of this finding is that Class I or Class II behaviour depends on the relative influence of the ‘elastic’ and the ‘inelastic’ portions of the test specimen. It is thus possible that the same rock material can exhibit Class I or

Class II type behaviour depending on the geometry of the test specimen. A further consequence is that whether or not a test specimen fails in a brittle, explosive manner is also dependent on the test specimen geometry and is thus equally not a material characteristic.

The statement as published in the introduction to the ISRM SM (ISRM 2007d), that force and load and also displacement and deformation are synonymous (in the context of inspecting laboratory test results) is fundamentally incorrect for the post-peak part of such curves. In the post-peak region the test specimens are of a compound nature consisting of unfailed fragments between cracks and ultimate failure planes. Strains calculated from measured deformations and the numerical values for the deformation characteristics calculated from such strains depend on the method of measurement. Deformation transducers average the deformations over the effective gauge length and thus the position of transducers relative to the failure planes can affect the resulting measurements significantly. In the extreme case e.g. when testing very high specimens the measured axial deformations may even represent the behaviour of ‘intact’ rock during unloading.

A further, important consequence of the experimental findings is that the practice of using values for the post-peak stress-strain response as input data for computer modelling without taking cognizance of the shape of the test specimens from which the values were evaluated is scientifically incorrect. Therefore, although reporting the results of tests in the post-peak region in the form of stress-strain curves is practical, their interpretations should be done with the necessary circumspection. For comparing the post-peak deformation behaviour of different types of rocks it is therefore important that specimens with a standard geometry should be tested.

It was shown in Section 3 that the strength of the Anorthosite tested in triaxial compression with a confining pressure of 10 MPa is not affected by the test specimen geometry except for short specimens with height to diameter ratios < 2.5 . This behaviour was to be expected and is perfectly in line with the standard

recommendation proposed by the ISRM. This conclusion is expected to be valid for other rock types as well as for different confining pressures.

The deformation behaviour in the pre-peak region of the stress-strain curves is also not affected by the test specimen geometry except when testing short specimens with height-to-diameter ratios < 2.5 . (Section 3) This behaviour was also expected and is again in line with the standard recommendation proposed by the ISRM.

For all the tests performed on the Anorthosite rock (Section 3), the mode of ultimate failure was that of a single failure plane. An explanation for this observed phenomenon is that all the excess elastic energy stored in the unfailed parts of the test specimen was effectively removed leaving only sufficient energy required to continue deforming the specimens at the pre-programmed rate. Such an explanation has not previously been reported and is therefore an important conclusion reached from the observations made during the testing programme. A notable consequence of this finding is that the degree of fragmentation depends on the success or otherwise of rapidly removing any excess elastic energy from the test specimen in addition to removing the elastic energy stored in the equipment. The phenomenon should be further researched for different types of rock. It is most likely that even when rocks with Class I type behaviour are tested using stiff servo-controlled equipment, the predominantly single plane mode of failure will result. However, this should be confirmed experimentally.

It was shown in Section 3 that the angle of the failure planes is dependent on the geometry of the test specimen for lower height-to-diameter ratios of less than 2.5. For height-to-diameter ratios of 2.5 and greater, the angle was found to be approximately constant at 24° . It must be stressed that this finding applies to the Anorthosite rock material when tested in triaxial compression with a confining pressure of 10MPa. It is expected that the value of 24° may differ for different types of rocks and also for different confining pressures. The phenomenon should be further investigated.

Quantifying the total energy and the energy per cubic meter from the areas under the load-deformation curves and stress-strain curves respectively was found to be useful when interpreting the results from post-peak testing. The total energy used when deforming and crushing the rock specimens with different height-to-diameter ratios was found to be dependent on the geometry of the test specimen. While this was to be expected the energy expended per unit volume was also found to be different. The quantity of energy released by the test specimens during the Class II behaviour in the post-peak region of the stress-strain curves was also found to be dependent on the height-to-diameter ratio. The findings again confirm the dependence of the specimen behaviour on the geometry in addition to the material properties.

The expression 'Class II type rock' should be avoided and may be replaced with expressions such as 'rock exhibiting Class II type behaviour'. When reporting results of tests, details of the experimental conditions such as specimen shape and size and confining pressure - if any - should always be reported. Reporting of the experimental details is anyway good practice and is also specified by ISRM and ASTM.

The artificial model material consisting of stabilized soil offers possibilities of further studying the post-peak properties of materials with Class I type behaviour. Test specimens of such materials can be made to standardized specifications resulting in uniform, consistent properties which should help to ensure repeatable test results. The model materials may be tested using 'soft' testing machines available in most test laboratories. Simple and inexpensive measuring equipment will be suitable for the tests and damage to such equipment is unlikely.

The stiffness of the test equipment has been shown to be the paramount factor determining the load-deformation path when swelling strain tests are performed. (Appendix D) Only the final values, after equilibrium conditions have eventually been reached, represent the material properties. In this study it has been explained

how the stiffness of the equipment can be calculated from the slope of the load-deformation measurements made during such swelling strain tests. Based on the observations made during the swelling tests it has been concluded that relaxation tests are equally affected by the stiffness of the test equipment. If a, hypothetically, infinitely stiff test apparatus could be used for such tests, the load would drop to zero as soon as any relaxation takes place and the experiment would be automatically terminated. On the other extreme, if an infinitely soft apparatus, for example a constant dead weight set-up is used, then typical creep curves will result. As was concluded from the swelling strain tests, the stiffness of the equipment used for relaxation tests can be calculated from the slopes of the resulting load-deformation curves. This important conclusion can also be applied to determine the stiffness of displacement-controlled and of servo-controlled loading equipment. Relaxation tests using specimens of suitable rock materials have to be performed for this purpose.

The method of programming servo-controlled testing machines by using a linear combination of stress and strain as applied by Okubo and Nishimatsu (1985) should be implemented. The method does not require an expensive circumferential transducer and has thus the potential of substantial savings. The method furthermore reduces the possibility of the transducer being accidentally destroyed. It does however have the disadvantage of not providing information on transverse strains.

The effect of transverse deformations should be further studied. It might give valuable information on the failure processes in the post-peak region. The transverse deformation modulus may even be a useful tool for assessing the brittleness of a material. This recommendation may read like a contradiction to the above suggestion of investigating the Okubo method, but will depend on the actual aim of a proposed test programme.

It is recommended that more attention should be given to work and energy determinations. This is expected to provide data allowing explanations for mode

of failure and particularly the fragmentation intensity. Furthermore, when inspecting results of cycling tests, the loss or apparent gain in energy should be carefully considered. The loading and unloading curves for perfectly elastic materials, whether linear or non-linear, are identical and there will be no gaps and no areas between successive cycles indicating that the energy spent and recovered remains constant. If the area of any gaps above stress-strain or load-deformation curves between successive loading cycles is larger than the area enclosed by the cycles it actually means that less energy is required during cycling tests than during single loading tests. Although the phenomenon of saving energy by cycling is unlikely, it is often observed in actual testing practice. Explanations should be sought in probable experimental errors such as the effect of hysteresis during the measurements.

To investigate the post-peak behaviour of very brittle rocks, for which even the most sophisticated testing equipment is not able to provide the necessary data, the use of a hybrid testing system should be investigated. Such a hybrid system could consist of a combination of stiffening elements, special load and deformation measuring transducers and servo-controlled testing machines.

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* The American Society for Testing and Materials (ASTM) publishes their ASTM standards annually. Each standard has a fixed designation number followed by the year of original adoption. Where applicable, the year of re-approval is added.

Appendix A

ADDITIONAL EXPERIMENTAL INFORMATION

ON

DETERMINATION OF THE POST-PEAK BEHAVIOUR OF A STABILIZED SOIL MATERIAL

a. Stress-Strain Curves

b. Photographs

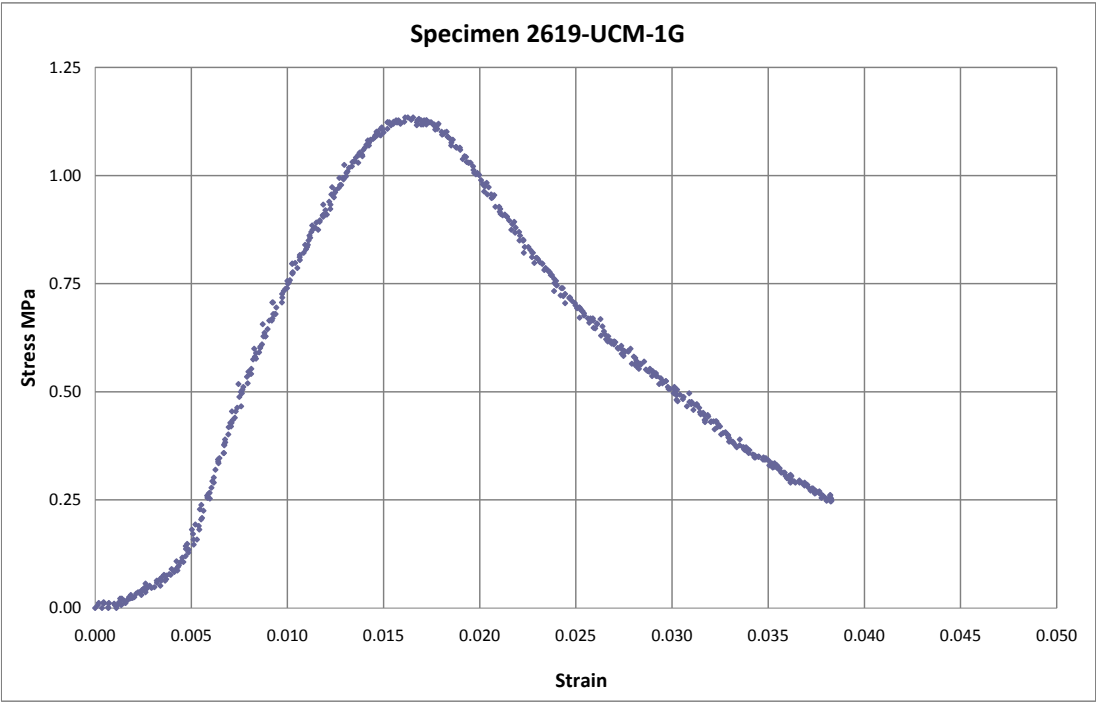


Figure A.1 Stress-strain curve for specimen 2619-UCM-1G

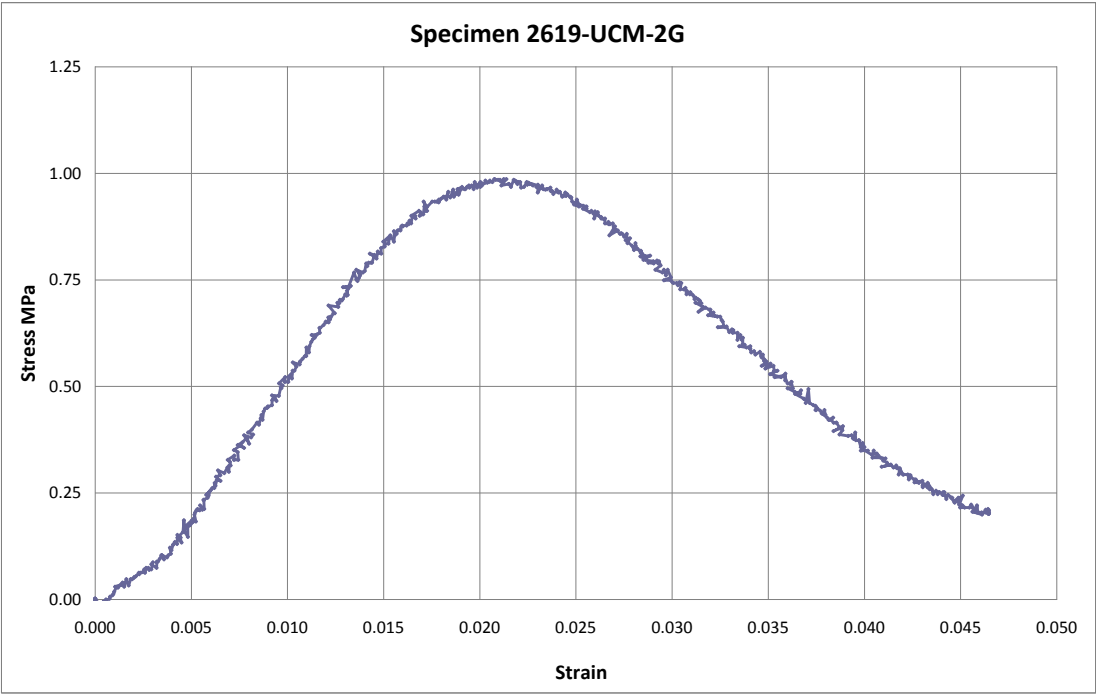


Figure A.2 Stress-strain curve for specimen 2619-UCM-2G

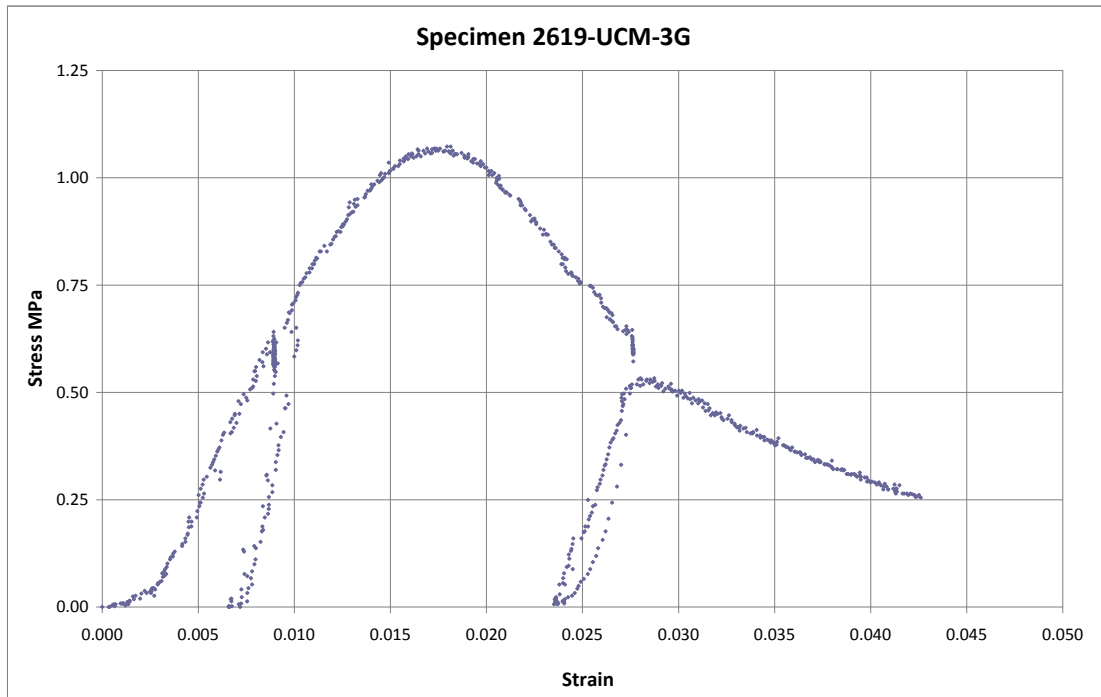


Figure A.3 Stress-strain curve for specimen 2619-UCM-3G

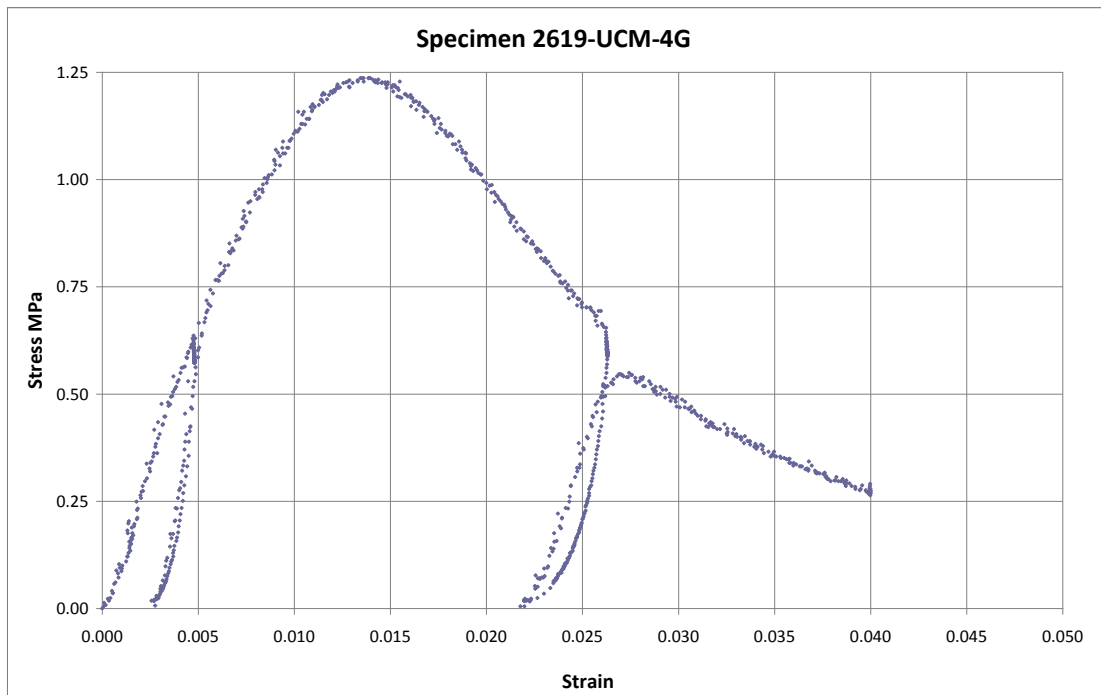


Figure A.4 Stress-strain curve for specimen 2619-UCM-4G

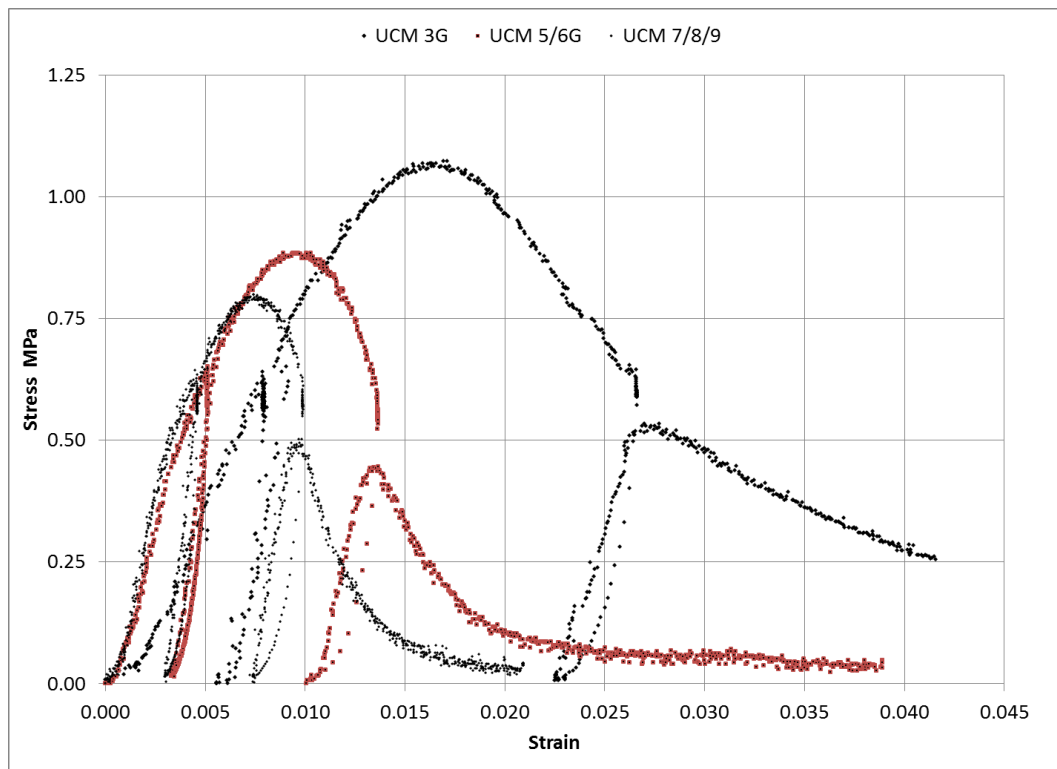


Figure A.5 *Stress-strain curve for specimens 2619-UCM-3G and combined specimens UCM 5/6G and UCM 7/8/9G*



Figure A.7 Test Specimen 2619-UCM-G2 during testing

Note: Predominantly vertical surface cracks (extension cracks).



Figure A.8 Test Specimen 2619-UCM-G4 after testing and removal of loose fragments.

Note: A predominantly diagonal failure plane may be noted.



Figure A.9 *Compounded specimen 2619-UCM-G5/6 during testing*

Note: Predominantly vertical surface cracks (extension cracks) not affected by the interface between the individual specimens.



Figure A.10 *Compounded specimen 2619-UCM-G5/6 after testing and after removal of loose fragments.*

Note: A predominantly diagonal failure plane.



Figure A.11 Compounded specimen 2619-UCM-G7/8/9 after testing and removal of loose fragments.

Notes: A continuous failure plane extending over the interface between constituent specimens.

For this very long specimen the predominant failure plane is not diagonal over the full length but its projection exits somewhere near the top of the lowermost constituent specimen.

APPENDIX B

TESTING OF ANORTHOSITE SPECIMEN UCM-01

B.1 Introduction

As described in the main part of this dissertation, a rock material with properties suitable for ensuring a reasonable chance of tracking the complete load-deformation curve in the post-peak region was required. Ideally the uniaxial compressive strength (UCS, σ_c) should be about medium according to the classification system by Deere and Miller (1966) as adapted to SI units. An extract from the Deere Classification is given below.

Table B1 Extract from Deere strength classification for intact rock

Very Low Strength	$10 \text{ MPa} \leq \sigma_c < 25 \text{ MPa}$
Low Strength	$25 \text{ MPa} \leq \sigma_c < 50 \text{ MPa}$
Medium Strength	$50 \text{ MPa} \leq \sigma_c < 100 \text{ MPa}$
High Strength	$100 \text{ MPa} \leq \sigma_c < 200 \text{ MPa}$
Very High Strength	$200 \text{ MPa} \leq \sigma_c < 400 \text{ MPa}$

The strength should be high enough to make sure that it will have Class II type behaviour while on the other hand it was essential that the full stress-strain behaviour could be successfully traced with the available test equipment. Anorthosite was expected to be a suitable rock material (Watson 2010) because it was expected to fulfill the following requirements:-

- Anorthosite is an igneous rock material and hence may be expected to provide repeatable results.
- The UCS was expected to fall into the abovementioned range.
- Anorthosite is relatively easy to obtain since it is present in the Bushveld Igneous Complex. It is abundant in the Mines around Rustenburg/Brits/Northam where it may be found above as well as below the Merensky Reef.

The aim of the preliminary test described in this appendix was therefore to establish whether the available Anorthosite rock is suitable for testing the premise of the research proposal.

Details of the origin of the Anorthosite are as listed in Table B.2.

Table B2 Origin of the Anorthosite rock material

Rock Material:	Spotted Anorthosite
Origin:-	Bushveld Igneous Complex Impala Platinum Mine Foot Wall – 10 shaft Between 1866/7 and 1866/8 South Approx. 1100 m below surface

B.2 Test Equipment and Procedure

The test was conducted in the rock testing laboratory of the University of the Witwatersrand. Their Amsler hydraulic testing machine with capacity of 2000 kN was used. The loading was controlled manually at a constant rate chosen to be somewhat slower than the standard rate as recommended by ISRM (2007, a). This was done to allow sufficient time for changing from loading to unloading and thus enable load cycling.

Axial and lateral strains were measured by means of strain gauges having 20mm gauge length. The data acquisition rate was set to record values in equal stress steps (actual average step 0.65 MPa). The curve of stress vs. axial strain was shown in real time on the computer monitor. The first unloading cycle was done when a stress level of about 70 MPa was reached. The second cycle was initiated when the stress-axial strain curve showed that failure was imminent.

B.3 Results

Axial, lateral and volumetric stress-strain curves are shown in Figure B.1.

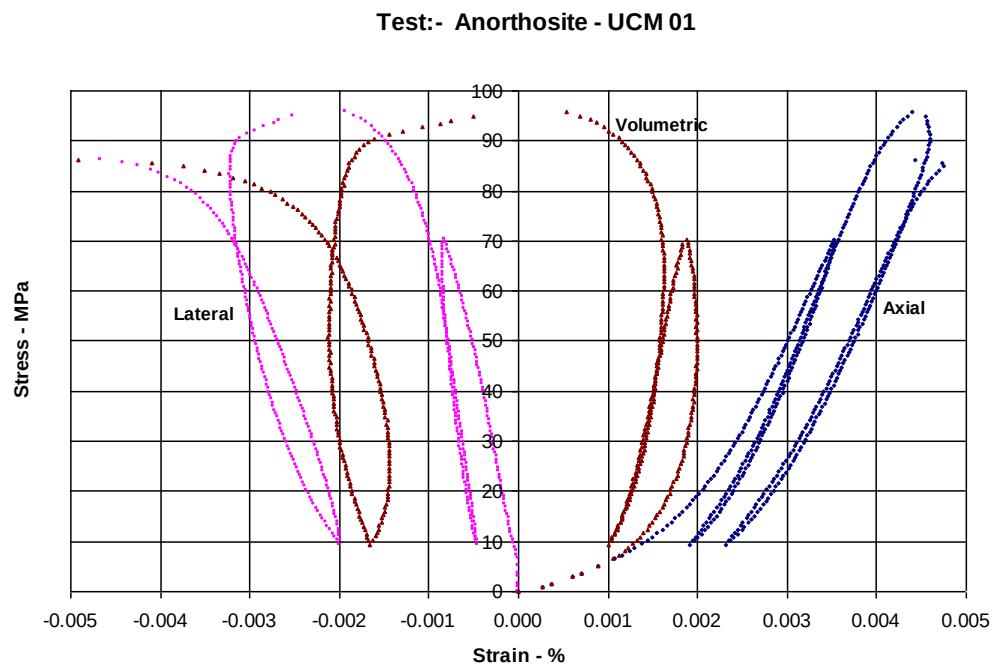


Figure B.1 Axial, lateral and volumetric stress-strain curves

The results obtained have been summarized in Table B.3

Table B3 Summary of Test Data

Specimen number:	UCM-01
Date of test:	2010-05-14
Diameter; height:	72.6 mm; 207.9 mm
Height-to-dia. ratio:	2.86
Mass; density:	2436 g; 2830 kg/m ³
Strength (UCS, σ_c):	97 MPa
$E_{\tan 50\%}$; $V_{\tan 50\%}$:	39 GPa; 0.51
Modulus Ratio (E/σ_c):	409
E Cycle 1 - unloading:	48 GPa
E Cycle 1 - reloading:	45 GPa
E Cycle 2 - unloading:	40 GPa
E Cycle 2 - reloading:	39 GPa

B.4 Observations

Inspection of the curves shows:-

- The general shape of the axial stress-strain curve is typical for what Watson (2010) describes as “strong nonlinear stress-strain behaviour”.
- Prior to reaching the peak of the stress-strain curve the axial stress-strain curve shows significant deviation from linearity.
- Similarly, but even more significantly, the lateral stress-strain curve clearly shows acceleration of the lateral strain rate.
- During the test it was possible to detect on the monitor that failure of the specimen was imminent, yet there was still sufficient time to allow for a second unloading cycle followed by a subsequent reloading cycle. It

should be noted here that the Amsler Machine is ‘soft’ and can only be adjusted manually.

- From the axial stress-strain curve it may be seen that load cycling (unloading followed by reloading cycles) does not significantly influence the subsequent stress-strain behaviour.
- The value for the Poisson’s Ratio is already >0.5 at 50 % of the strength. This also means that the minimum volume of the specimen is reached around 49 MPa and in addition that the stress level at “Onset of unstable fracture propagation” (Bieniawski et al, 1967b) is at about 49 MPa.
- The relatively large Poisson’s ratio means that there is high lateral strain. This in turn is expected to result in sufficiently large signal strength for feedback control for the research tests to be conducted in the Wits MTS test machine.

B.5 Conclusions

The Strength of the Anorthosite falls in the upper region of a ‘medium’ strong rock. It may thus be expected to be a rock material exhibiting Class II type behaviour.

It may furthermore also be expected that stress-strain behaviour can be successfully monitored in the post-peak region.

The spotted Anorthosite was therefore accepted for the research tests intended for testing the premise of this dissertation.

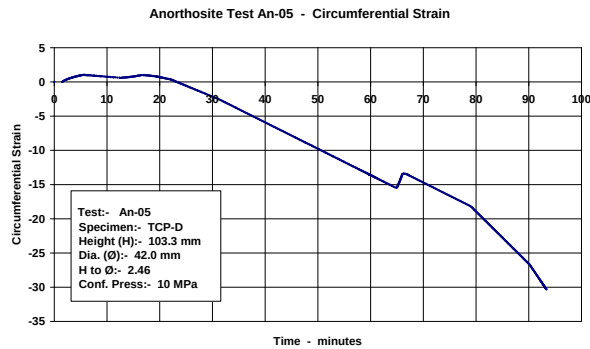
APPENDIX C

ADDITIONAL EXPERIMENTAL INFORMATION

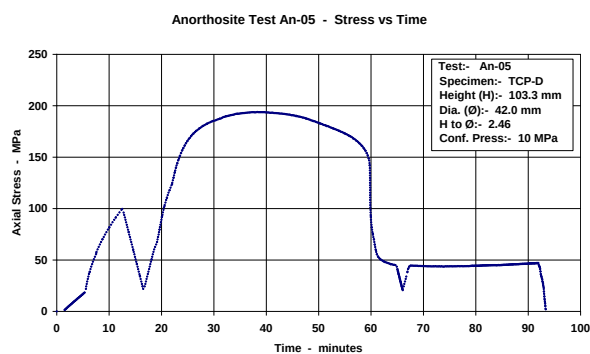
ON

DETERMINATION OF THE POST-PEAK BEHAVIOUR OF SPOTTED ANORTHOSITE

Appendix C – Annexure 1

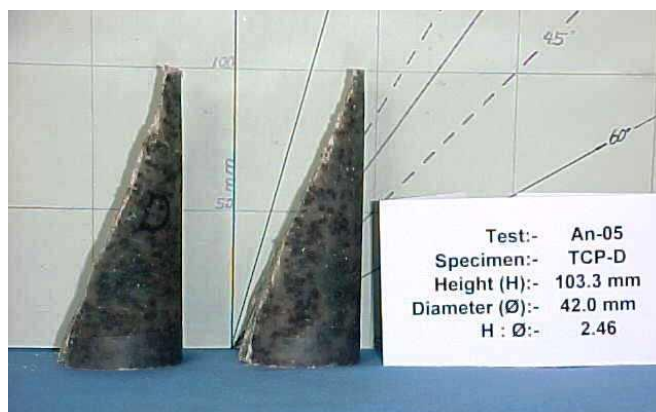
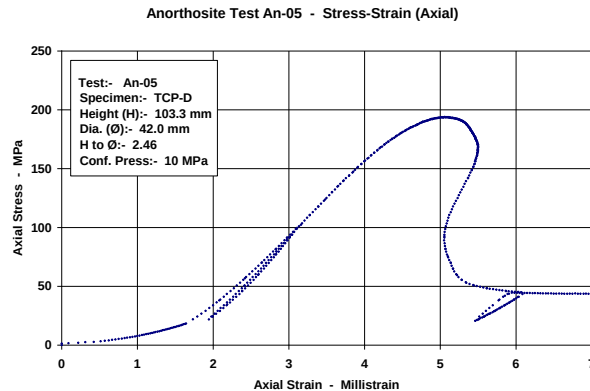


Test:- An-05
Strength (MPa):- 193.9
Tan 50% E (GPa):- 64
M (GPa):- 129
Failure Angle:- 24.0°

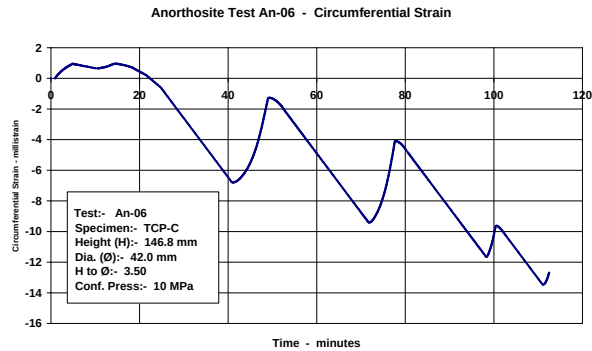


Notes:-

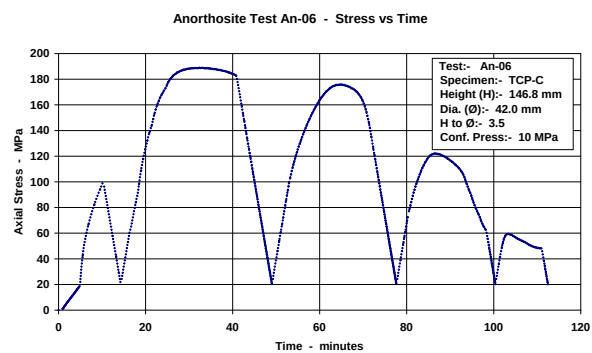
1. At 80 and 90 min increase in strain rate caused slight increase in resistance instead of levelling off at end of test.
2. Specimen strained to >>0.7% resulting in puncture of sleeve and oil contamination, hence dark colour of specimen.



Appendix C – Annexure 2

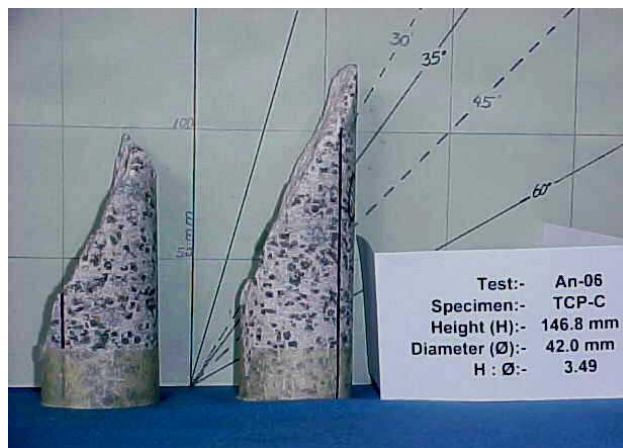
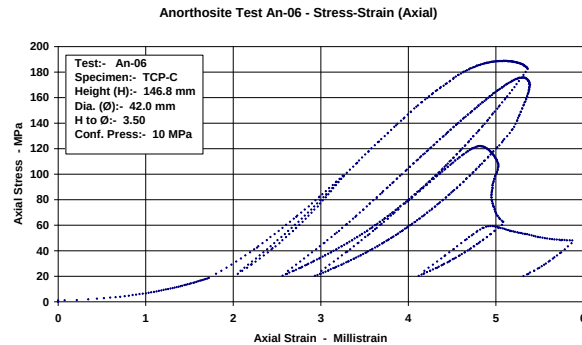


Test:- An-06
Strength (MPa):- 188.9
Tan 50% E (GPa):- 64
M (GPa):- 126
Failure Angle:- 23.7

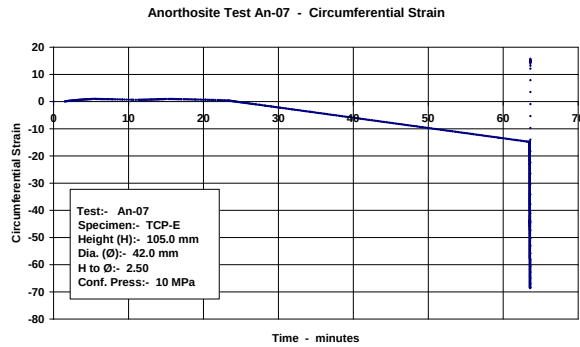


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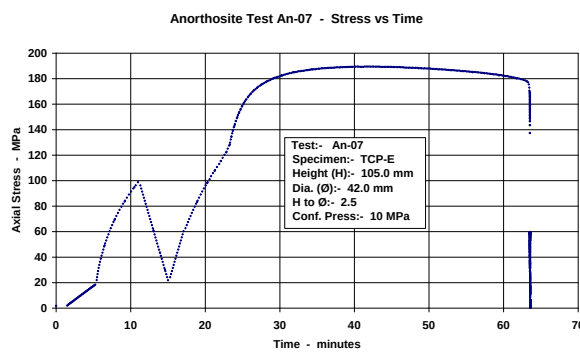
1. First attempt of multiple cycling was successful, but possibly data in critical phase of post peak curve may have been lost.
2. For energy calculations the two additional cycles were removed and the gap was closed by an artificial smooth curve.



Appendix C – Annexure 3

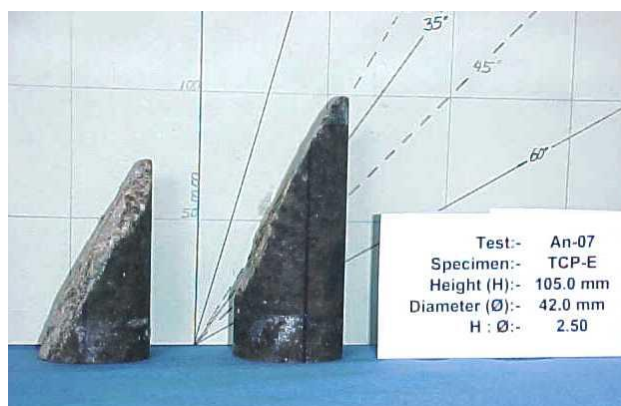
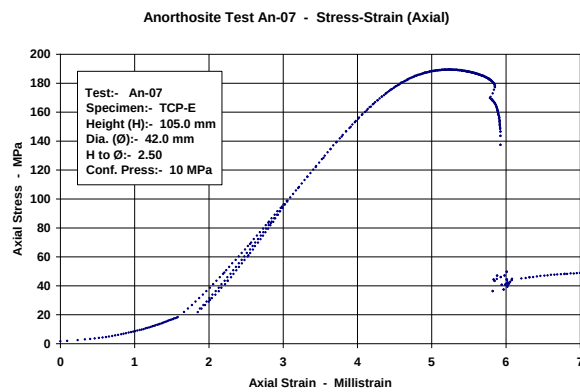


Test:- An-07
Strength (MPa):- 189.5
Tan 50% E (GPa):- 60.0
M (GPa):- event
Failure Angle:- 25.8

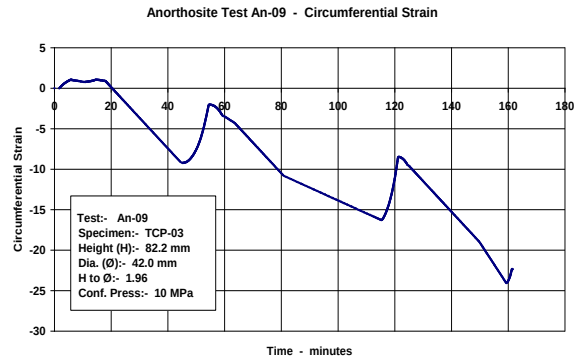


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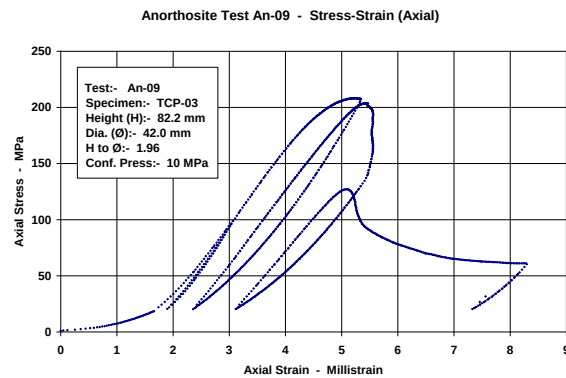
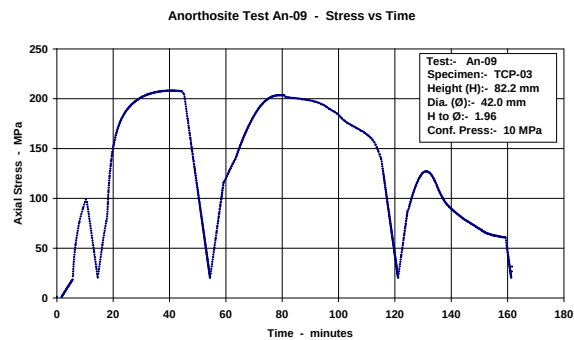
1. Rapid unloading due to loss of control on changeover to different mode of control.
2. Test specimen dark due to oil contamination because of ruptured sleeve.



Appendix C – Annexure 4



Test:- An-09
Strength (MPa):- 208.2
Tan 50% E (GPa):- 65
M (GPa):- 194
Failure Angle:- 27.1

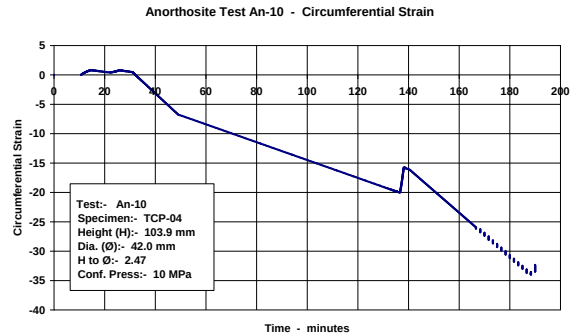


Notes:-

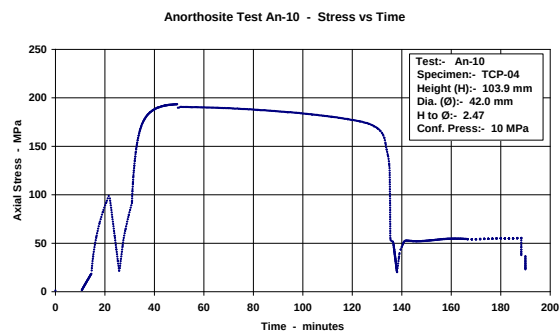
1. At 80 minutes a decrease in circumferential strain rate resulted in a minor decrease in resistance.
2. The second unloading – reloading cycle was close to the point of peak strength and may have affected the post peak behaviour and energy calculations.
3. The third unloading cycle affected energy calculations.



Appendix C – Annexure 5

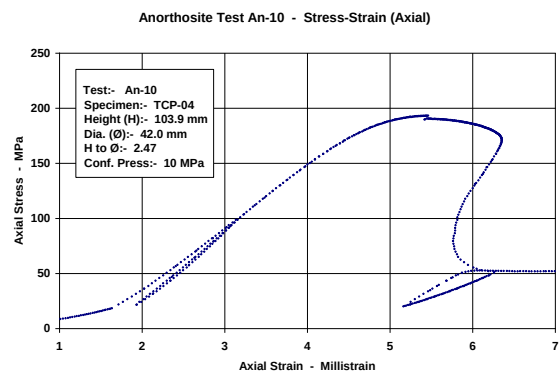


Test:- An-10
Strength (MPa):- 193.4
Tan 50% E (GPa):- 58
M (GPa):- 115
Failure Angle:- 24.6

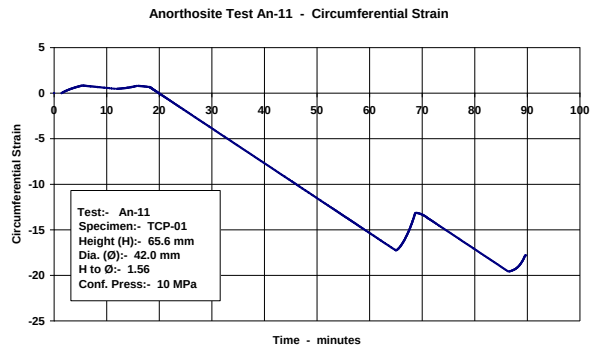


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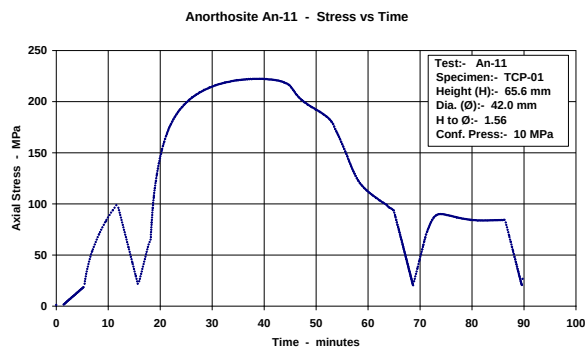
1. At 50 minutes a decrease in circumferential strain rate resulted in a minor reduction in resistance.



Appendix C – Annexure 6

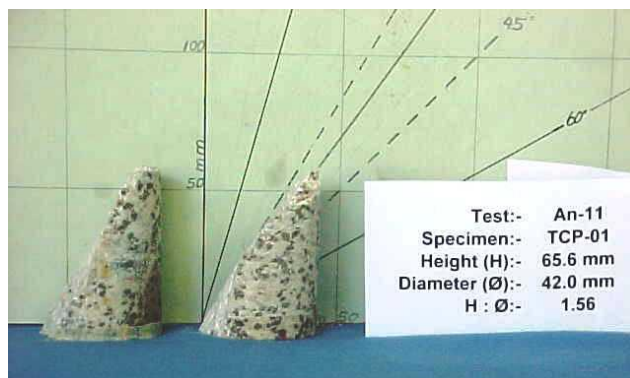
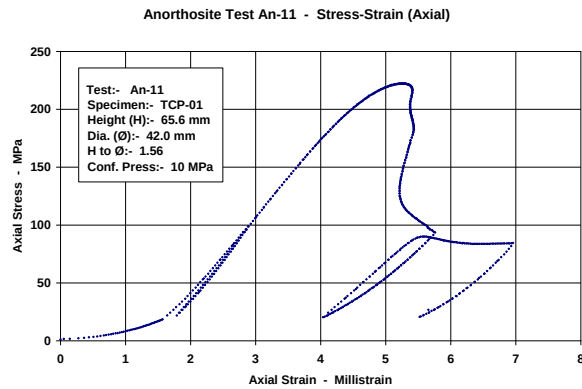


Test:- An-11
Strength (MPa):- 222.3
Tan 50% E (GPa):- 67
M (GPa):- 198
Failure Angle:- 32.6

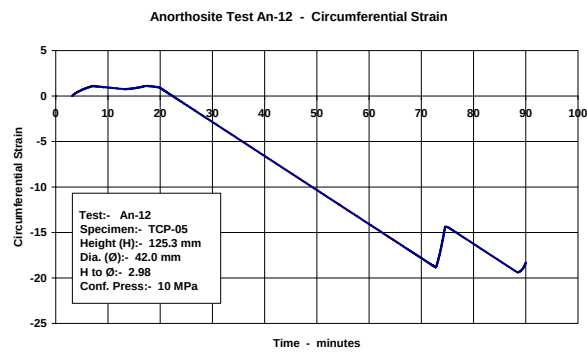


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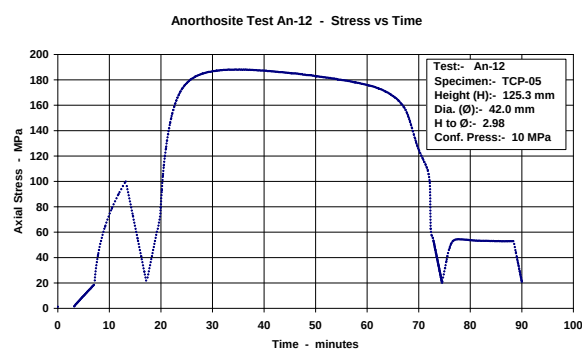
1. Signs of double cone development at specimen ends.



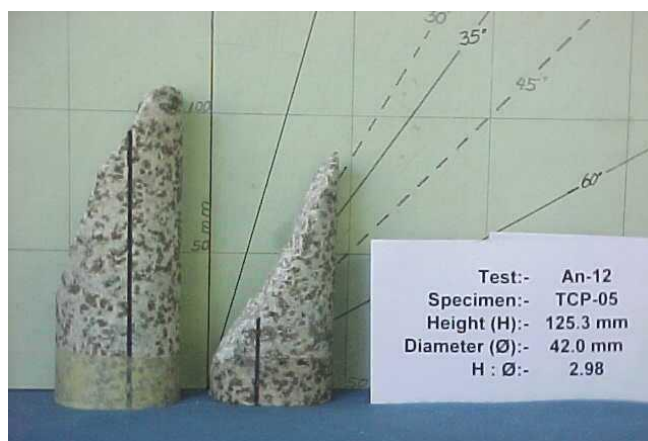
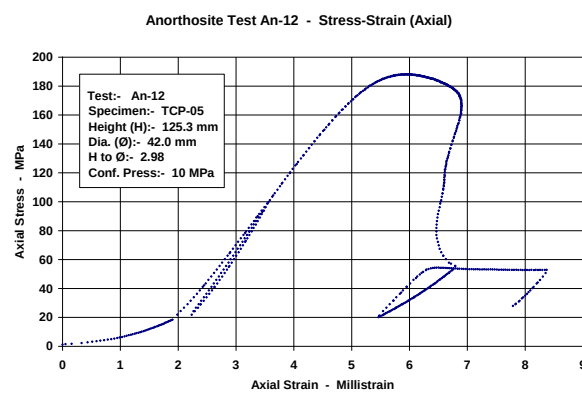
Appendix C – Annexure 7



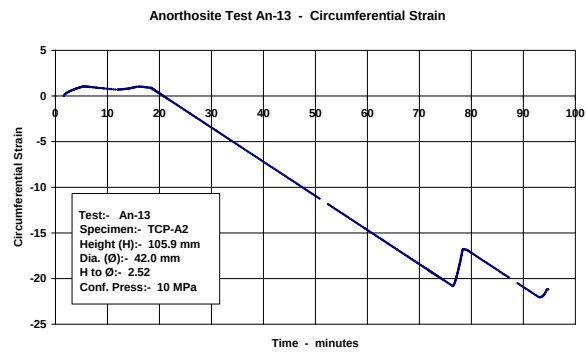
Test:- An-12
Strength (MPa):- 188.2
Tan 50% E (GPa):- 54
M (GPa):- 113
Failure Angle:- 24.1



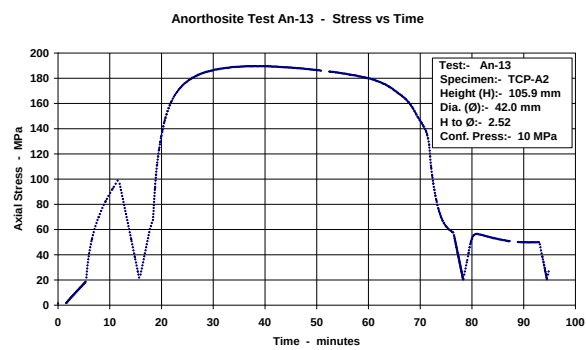
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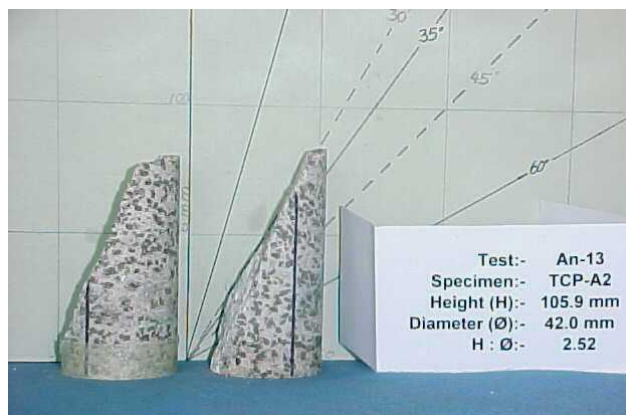
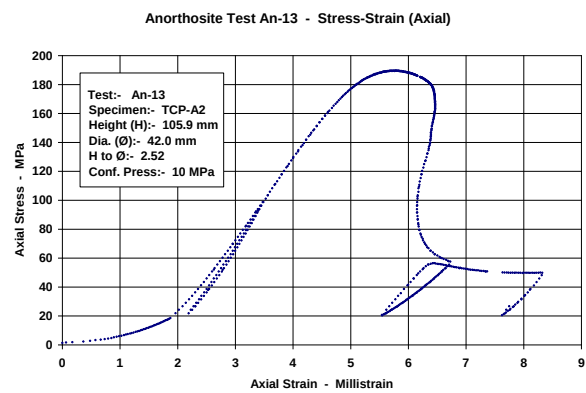
Appendix C – Annexure 8



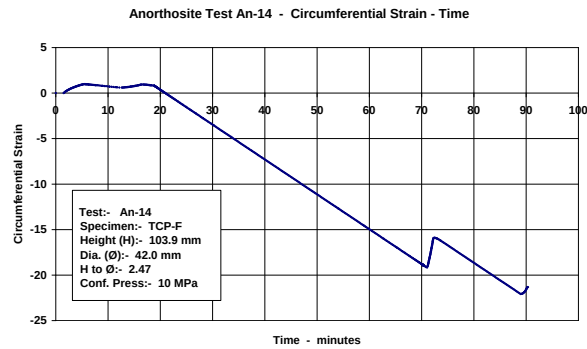
Test:- An-13
Strength (MPa):- 189.7
Tan 50% E (GPa):- 56
M (GPa):- 133
Failure Angle:- 24.8



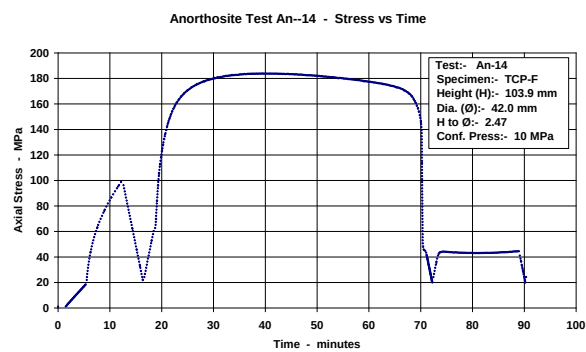
Notes:-



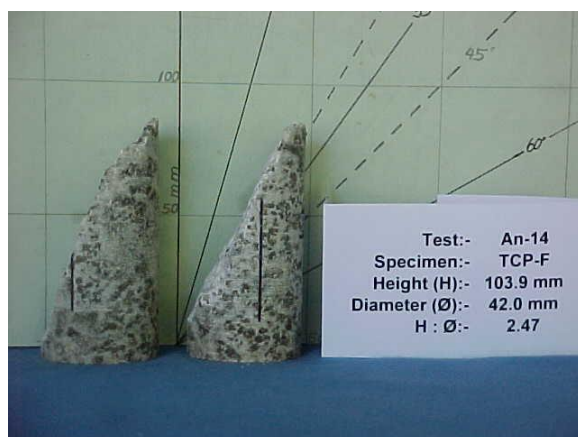
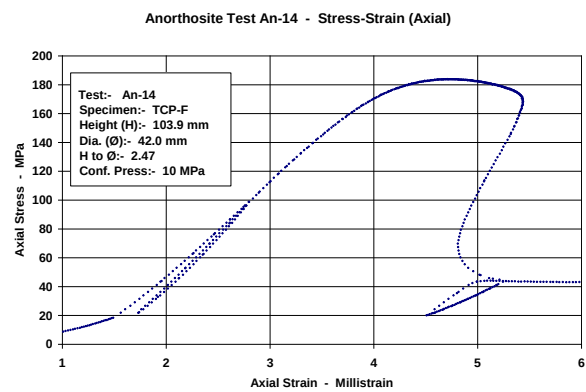
Appendix C – Annexure 9



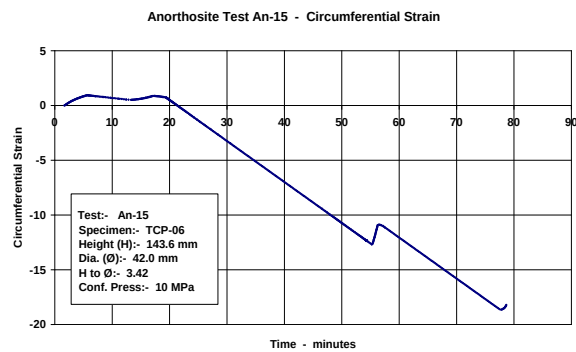
Test:- An-14
Strength (MPa):- 183.9
Tan 50% E (GPa):- 67
M (GPa):- 129
Failure Angle:- 24.8°



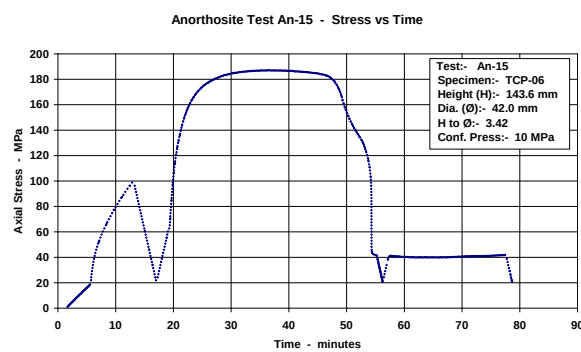
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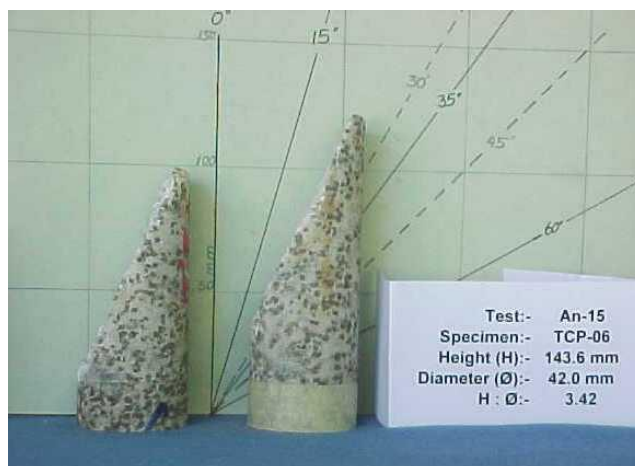
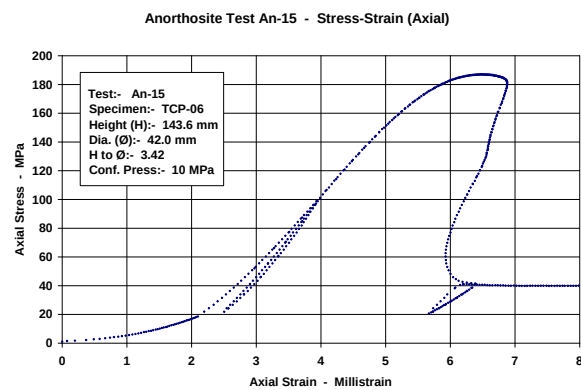
Appendix C – Annexure 10



Test:-	An-15
Strength (MPa):-	187.1
Tan 50% E (GPa):-	54
M (GPa):-	91
Failure Angle:-	23.9



Notes:-



APPENDIX D

PARTIALLY RESTRICTED SWELLING STRAIN TESTS

D.1 INTRODUCTION

The work briefly summarized below was previously done by the Author while employed at the CSIR, however, it has not previously been published.

When performing tests for the determination of axial free swelling strain the test specimen is allowed to absorb water, no restriction is allowed and the resulting change in axial dimension is measured and evaluated. The results are usually reported as swelling strain-time graphs. (ISRM, 2007c)

On the other hand, when the objective is to measure the swelling pressure, then the swelling strain developing during hydration is compensated for by applying the force necessary to restore the strain to the original zero value. The method results in stepped stress-time curves. Once stability has been reached, the stress eventually required to restore the original specimen dimension is taken as the swelling stress.

D.2 DESCRIPTION OF TESTS AND OF TEST RESULTS

In addition to standard swelling stress and swelling strain determinations described above, additional tests were performed using the equipment for the swelling pressure tests. During the additional tests a deviation from the standard method was to not to effect any adjustments. Swelling strain and swelling

pressure were allowed to develop and were recorded against time. Examples of such curves are shown in Figures D.1 and D.2 respectively. Both graphs show the variables reaching final maximum values asymptotically with time. The duration of the test was about 550 hours or 23 days. It should be noted that for such long duration precision tests a laboratory with constant temperature was available.

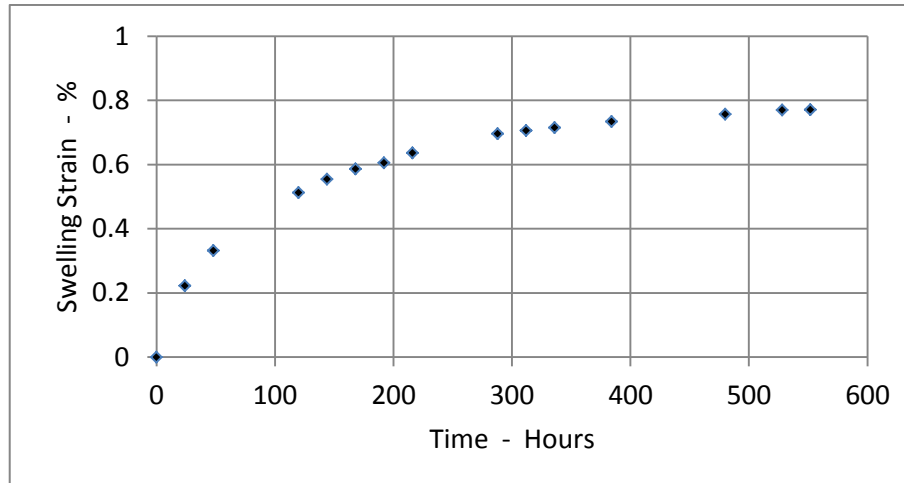


Figure D.1 *Development of swelling strain with time*

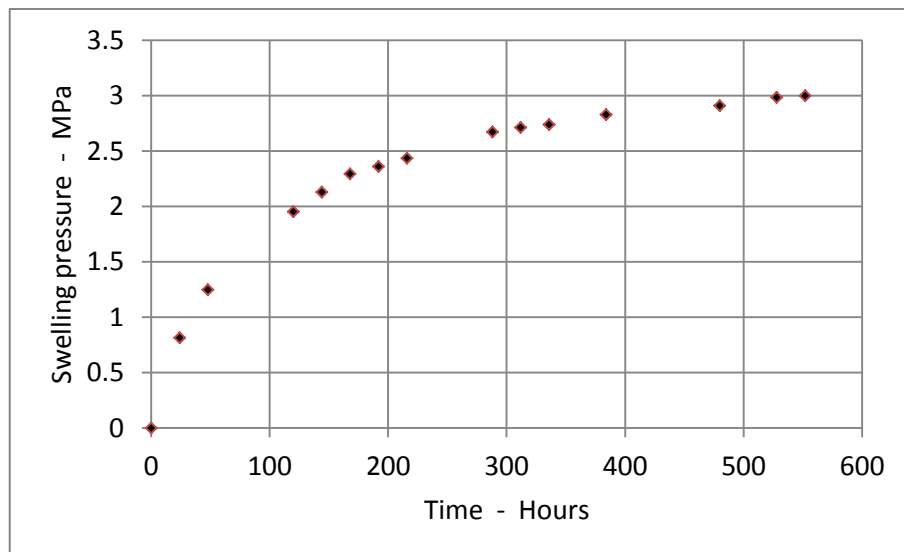


Figure D.2 *Development of swelling pressure with time*

When plotting the swelling stress vs. swelling strain a perfectly linear relationship was established. See Figure D.3 below. The observations have been confirmed for other specimens and different rock materials.

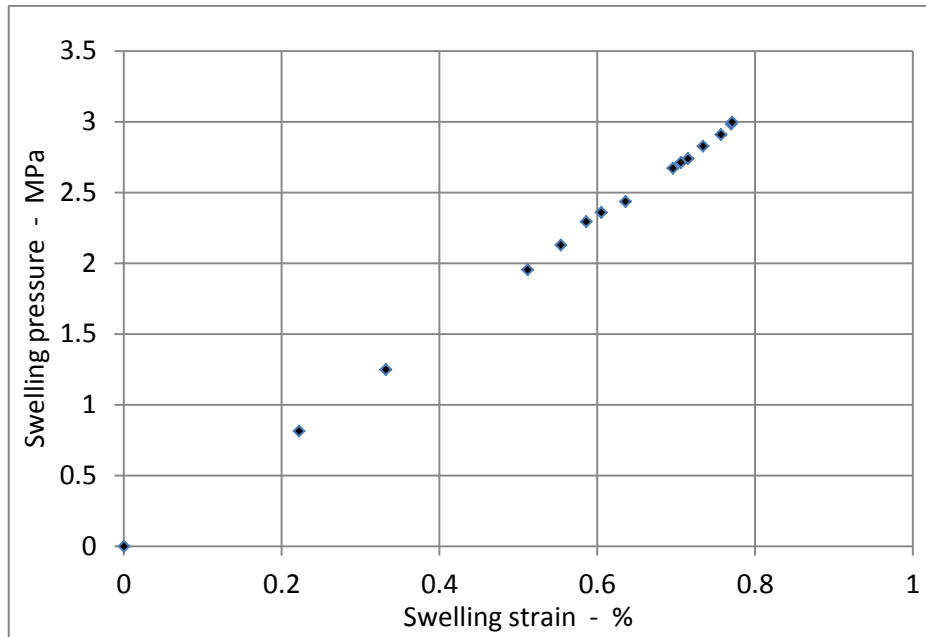


Figure D3 *Swelling pressure vs swelling strain*

D.3 DISCUSSION OF THE OBSERVED PHENOMENON

- During the swelling process the length of the rock test specimen gradually increases, but its increase is restricted by the resistance of the equipment.
- Due to the restriction offered by the equipment a swelling pressure develops.
- The force resulting from the swelling pressure acts against the equipment and causes it to deform.
- The deformation of the equipment allows the test specimen to swell further.
- The process continues until equilibrium conditions are reached.
- The interaction between the force (resulting in stress) and the equipment deformation (resulting in strain) is proportional to the effective stiffness of the test equipment.
- The slope of this relationship shown in Figure D3 is thus a direct function of the stiffness of the equipment and is not the property of the rock tested.

- On the other hand, the final asymptotic value for the stress and the strain do represent the property of the rock.
- By calculating the load increment (ΔN) and the increment in deformation (Δm) from the stress and strain the stiffness $K = \Delta N / \Delta m$ in Newton per meter can be calculated.

D.4 CONCLUSIONS ON SWELLING STRAIN TESTS

The stiffness of the test equipment determines the slope of the swelling pressure-swelling strain curve and not the rock specimen.

The effective stiffness of the test apparatus can be calculated from the slope of the swelling pressure vs swelling strain curve.

By performing tests using equipment with different stiffnesses the relationship between swelling pressure and swelling strain can be established.