

School of Mining Engineering



UNIVERSITY OF THE
WITWATERSRAND,
JOHANNESBURG

**OPTIMUM DEPTH FOR THE INTRODUCTION OF
CRUSH PILLARS AT IMPALA PLATINUM MINE.**

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

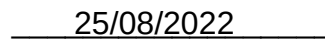
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ABSTRACT

Impala Mine planned to introduce 4 m x 2 m crush pillars on the UG2 Reef, and were interested in finding the depth where such pillars can be safely used. These pillars have the benefit of increasing the extraction ratio whilst reducing the risk of pillar bursting. The mine provided rock samples composed of four rock types namely: anorthosite, chromitite, pegmatoid and pyroxenite for geomechanical testing. In addition, a database of previous tested rock specimens was provided. The rock samples provided for testing were used to determine post-peak behaviour under triaxial loading conditions, as well as Uniaxial Compressive and Brazilian Tensile strengths. During testing it was apparent that the testing machine was showing incorrect deformation readings. Investigations showed the problem to be the axial transducer, and it was replaced. Subsequently, the results of the correct laboratory tests were used as input parameters for a FLAC3D numerical model. Two scenarios for the crush pillar were modelled. Scenario 1 assumes the presence of a 2 m siding and scenario 2 assumes the pillar is right next to the gully. The model assumed repeating geometries of pillars and the results suggested peak strengths of 104 MPa and 79 MPa for scenario 1 and 2, respectively, and a residual strength of about 12 MPa for both scenarios. The investigations showed that the 4 m x 2 m crush pillars can be safely introduced at depths of 663 m and 476 m for pillars with and without a siding, respectively. The pillars can be introduced at shallower depths (from 357 m for pillar with a siding and 272 m for the one without a siding) with caution as the available stress may not be sufficient.

DEDICATION

To the almighty God and my parents Mr and Mrs Maphosa

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LIST OF SYMBOLS

Stress	σ
Shear stress	τ
Strain	ε
Young's modulus	E
Poisson's ratio	ν
Cohesion	C
Internal friction angle	ϕ
Plastic shear strain	ε^p
Dilation angle	ψ
Pillar height	h
Effective pillar height	h_e
Pillar width	w
Pillar effective width	w_e
In situ cube strength	K_i
Pillar width strengthening parameter	α
Pillar height strengthening parameter	β
Pillar width-to-height ratio	w/h
Uniaxial compressive strength	δ_{ci} or UCS
Material constant	m_i

1 INTRODUCTION

The research in this project investigates the optimum depth at which 4 m x 2 m crush pillars can be introduced in the Upper Group 2 (UG2) reef at Impala Rustenburg mine. To achieve this objective, a series of triaxial tests were done to determine the post-peak behaviour on the supplied samples. The samples were composed of four rock types namely; anorthosite, chromitite, pegmatoid and pyroxenite. For continuity, uniaxial compressive tests and indirect tensile tests were also performed. These tests provided the necessary parameters to calibrate a FLAC3D numerical model. The aim of the model was to determine the stress conditions required for the crush pillar to work effectively.

1.1 Bushveld complex

The Bushveld Complex is one of the world's largest igneous layered intrusions and it hosts most of the platinum group metals on the planet. It also hosts other metals like iron, tin, nickel and gold. Chromium and vanadium seams occur parallel to the platinum orebodies. Chromium seams are normally situated in the footwall and vanadium in the hangingwall (Watson, 2010). The Merensky reef, UG2 and the Platreef are the only three platinum bearing orebodies known to be economically viable in the complex (Labuschagne et al., 2015). The most exploited platinum ore bodies in the complex are the Merensky and UG2. The Bushveld Complex is situated in the northern part of South Africa, north of the city of Pretoria. Figure 1.1 shows a map of the Bushveld Complex.

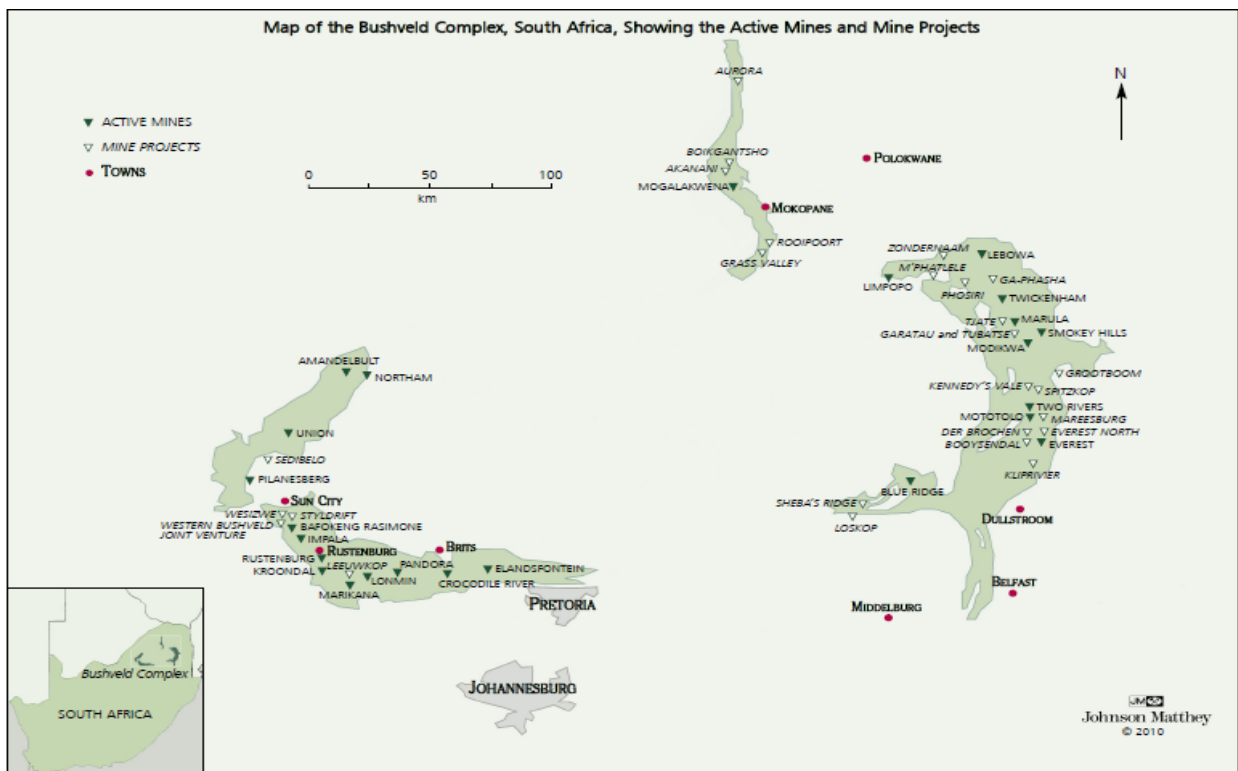


Figure 1.1 Map of the Bushveld Complex, showing the different limbs (the eastern, western and northern limbs). Major towns are marked in red and active mines (including mine projects) are marked in green (Cawthorn, 2010).

1.1.1 UG2 geology

The thickness of the UG2 reef varies between 0.4 m -2.5 m (0.5 m – 0.7 m at Impala but rarely reaches thickness over of 1 m). The UG2 reef is generally characterised by a pegmatoidal pyroxenite footwall, a main layer which is a combination of two or more (up to 9) chromitite layers overlain by melanoritic rocks that contain a thin chromitite leader and triplets (very thin chromitite layers). At Impala the immediate hangingwall above the UG2 Chromitite seam is typically comprised of a 6 m – 9 m thick layer of Pyroxenite. This layer is followed by anorthosite linked by a gradational contact. Chromitite layers (Intermediate Chromitite Layer (ICL), the Leader and Triplets) present in the pyroxenite, represent planes of potential parting. These planes occur from a few centimetres into the

hangingwall up to 4 m. A general stratigraphy column showing the typical sequence of rock types above and below the UG2 seam is provided in Figure 1.2. The dashed rectangle shows the rock types that were provided by the mine and used for numerical modelling presented in Chapter 4.

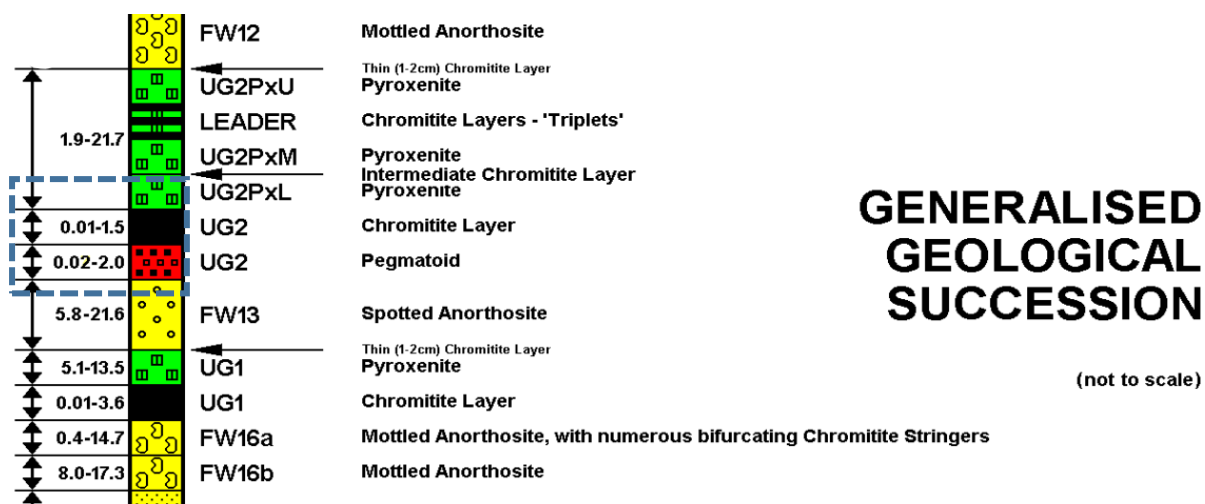


Figure 1.2 UG2 reef stratigraphy

Normal and reverse faults are encountered on the UG2 reef, with reverse faults being the most common (Impala Mine Rock Engineering Department, 2017). The strike of most faults tends towards the reef strike. Geological features like potholes and dykes are prevalent throughout the lease area and these structures contribute to geological losses. Potholes are oval structures where the reef elevation suddenly drops within the confines of the oval. On the UG2 horizon, the presence of potholes is usually indicated by the thinning of the chromitite layer and footwall lithologies and the absence of pegmatoid in the footwall.

1.2 Research background

The research was conducted at Impala Platinum Mine located in the Western Lobe of the Bushveld Complex. The mining operation is situated 30 km North of Rustenburg in the North West Province of South Africa. Figure 1.3 shows the location of Impala mine in relation to other mines on the Western Bushveld Limb.

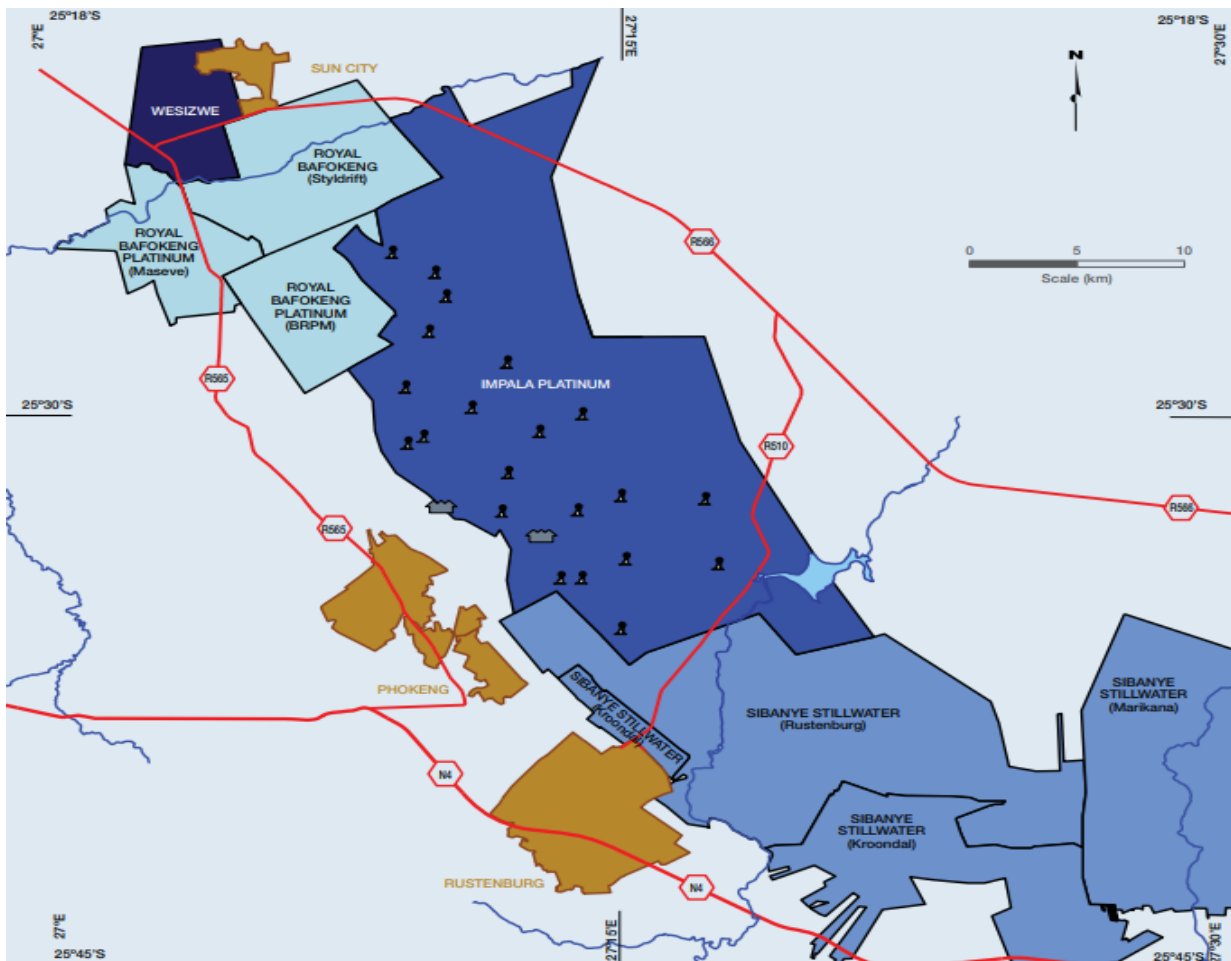


Figure 1.3 Location of Impala mine (Implats, 2018)

Impala mine is the world's second-largest producer of platinum group metals (PGM). The mine produces approximately 750 thousand ounces of refined platinum plus other PGMs

per annum. The Bushveld Complex comprises of alternating pyroxenite, chromitite, norite and several layers of anorthosite rock types. At Impala Platinum Mine, these layers have an average dip of 9 degrees towards the north-east. Two orebodies are currently mined, namely:

- Merensky Reef; and
- UG2 Reef.

The Merensky is almost mined out. In the north, the UG2 layer is 60 m below the Merensky Reef and the distance between the two reefs increases to 130 m in the south. The general strike direction of the layering is north-northwest to south-southeast but local variations can cause the strike direction to change. The average UG2 reef width is 0.91 m (Impala Mine Rock Engineering Department, 2017). The immediate hangingwall and footwall of the UG2 reef stope is pyroxenite and spotted anorthosite, respectively. The depth of the UG2 workings ranges between 30 m and 1400 m.

Underground mining operations are conducted from 12 shafts with two new shafts in ramp-up phase. Most of the mining is done using a scattered mining method with selective mining, and geological losses are left unmined. However, a limited amount of stoping is also done using mechanised room and pillar. The regional support strategy comprises barrier pillars together with geological losses. In the stopes, pillar support consists of non-yielding pillars, yield pillars and crush pillars. Figure 1.4 shows typical stress-strain behaviour of in stope pillars for the Merensky reef. Points Y and C marked on Figure 1.4 indicate the operating points of yield and crush pillars, respectively.

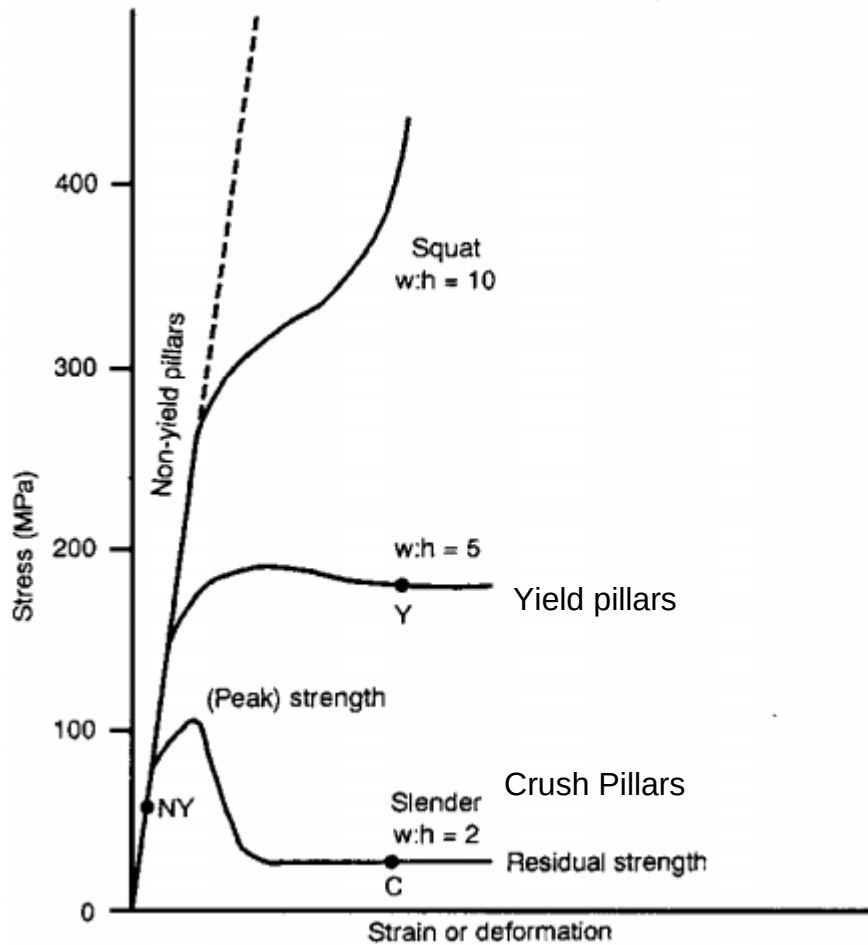


Figure 1.4 Typical in stope pillar stress-strain behaviour (Jager and Ryder, 1999)

1.3 Aims of the research

Currently the mine is using 6 m x 3 m yield pillars but plans to introduce smaller 4 m x 2 m crush pillars on the UG2 reef and are interested in determining at what depth these pillars can be introduced. Crush pillars have the benefit of improving the extraction ratio and do not present a burst risk once they reach the residual strength. However, crush pillars can be prone to violent failure if they do not fail close to the face (Watson, 2010). The available stress at the face must exceed the peak strength of the pillar otherwise crushing will not take place and the pillar becomes a hazard in the back area.

The objective of this research is to determine the depth at which crush pillars can be introduced. However, depth is not the only metric to use when deciding when to introduce pillars. In general, the available stress at the face determines whether the pillar will fail as it is cut. A particular depth does not guarantee that the stresses at the face will be sufficient to initiate failure of the pillar. More focus was therefore placed on determining the peak strength and the minimum available stress required for the 4 m x 2 m pillar failure. This was achieved by conducting laboratory tests and setting up a series of numerical models in FLAC3D to estimate the strength and available stresses.

1.4 Research summary

Chapter 2 reviews work done by previous researchers on post-peak behaviour of rocks. This includes triaxial testing using servo-controlled rock testing machines and factors that affect post-peak behaviour. The chapter also reviews previous investigations done on crush pillars. The history of crush pillars and the factors that affect their performance are described in this chapter.

Chapter 3 describes the methodology used during rock testing and the challenges that were faced whilst performing the rock tests. The testing procedures for triaxial (involving post-peak), Uniaxial Compressive Strength test (UCS) and indirect tensile test (Brazilian test) are described in this chapter. Procedures used in investigating issues pertaining to the testing as well as the solutions to these issues are also defined. The results from all the rock tests that were performed are presented.

Chapter 4 presents the numerical models. FLAC3D was used for numerical simulation of pillar strength and the determination of the available stress. A triaxial numerical model

was setup in FLAC3D with the objective of reproducing stress-strain curves obtained in Chapter 3. The calibrated model was subsequently used to simulate crush pillar behaviour. The objective of this model was to estimate the stress requirements of the 4 m x 2 m crush pillar. Two scenarios for the crush pillar were modelled. Scenario 1 assumes the presence of a 2 m siding. Scenario 2 assumes the pillar is right next to the gully. The chapter describes the strength and deformation characteristics of the pillar.

2 LITERATURE REVIEW

This chapter provides a literature review on post-peak rock testing and crush pillars. The role of machine stiffness and servo control in investigating post-peak investigations is discussed. In addition, the chapter describes the history of crush pillars and factors that affect their performance.

2.1 Sample testing

2.1.1 Calibration

Calibration refers to the setting of the origin and choice of scale to be used by a measuring instrument against a traceable reference or standard (Dawkins et al., 2001). This is important as it ensures the collection of accurate data and allows a comparison of collected data with other available data using the same standard. Over time the accuracy of a rock testing machine may change. This can be caused by wear and tear or the equipment standing idle for extended periods. Equipment calibration at recommended intervals is vital to minimise uncertainty and allow the equipment to give reliable, accurate and repeatable measurements.

2.1.2 Stiffness

Description

Stiffness is the ability of a material (in this case rock) to resist deformation when exposed to a stress (Hudson et al., 1972).

Implications of test machine stiffness on rock test

According to Hudson et al., (1972) a testing machine is characterised as either soft or stiff with respect to a given rock specimen. During testing, both the specimen and the machine deform as the load increases (Salamon, 1970). Salamon, (1970) observed that the equilibrium between the testing machine and the sample remains stable if the machine is unable to induce further displacement in the specimen without a supply of additional external energy. This is in line with observations made by Cook, (1963) which led him to conclude that the violent, brittle behaviour observed during testing, was due to excess energy stored in the machine (Cook, 1963). This resulted in the design of stiff testing machines. To obtain a complete stress-strain curve, the following condition must be met throughout the test to avoid abrupt violent failure (Hudson et al., 1972).

$$\textit{Stiffness of the machine} > \textit{the post peak stiffness of the sample} \quad (2-1)$$

The post-peak stiffness of the sample depends on factors such as length, loading-area, confinement, presence of discontinuities, etc. This means that the post-peak stiffness of a material is not an intrinsic property of the material (Beruar et al., n.d.).

Investigations done by Wawersik and Fairhurst, (1970) showed that the post-peak behaviour exhibited by rock samples can be classified into two distinct classes, Class I and Class II. The investigations also showed that high machine stiffness (which facilitates fracture control) on its own is not always sufficient to obtain a complete stress-strain curve (Wawersik and Fairhurst, 1970). Most rocks exhibit Class I strain softening (post-peak behaviour) where the tangential modulus is negative. The behaviour exhibited by a specimen can be affected by the depth at which the specimen was obtained. Specimens

obtained by Watson (2010) at depths greater than 1000 m at Impala Platinum exhibited a different class compared to specimens obtained from the same mine at depths less than 1000 m for the same rock type.

2.1.3 Class I post-peak behaviour

Rocks that exhibit class I behaviour require work to be done on them to induce further deformation. Fracture propagation is stable (Wawersik and Fairhurst, 1970) provided that the highest absolute value of post-peak stiffness is less than the machine stiffness (Hudson et al., 1972). Typical Class I behaviour of rock is that both the axial and lateral strain continuously increase during the deformation cycle (Oniyide, 2015). This means either axial or lateral strain can be set as the control to trace out the complete stress-strain curve. When a soft testing machine (machine that accumulates excessive strain energy in its components) is used on a Class I rock, the accumulated strain energy will exceed the energy required by the rock to propagate the fracture, resulting in violent failure (Vogler, 2014). However, if a stiff testing machine is used, the system (machine and rock sample) will remain in equilibrium as the elastic strain energy stored in the machine is less than the energy required by the rock sample for failure propagation. Figure 2.1 shows a typical Class I stress/strain curve with shaded regions indicating energy supplied by the testing machine and energy required to deform the rock specimen (Vogler, 2014). From Figure 2.1, additional energy would have to be supplied to continue testing from A to B. The shaded area (1+2+3) represents the supplied energy by the machine from O to B. The energy supplied by the machine to reach peak at A is represented by area (1+2). Area (1) represents the non-recoverable energy. This includes energy required for crack formation and crack propagation, plastic deformation, heat, extension, etc. Area (2) shows

the elastic energy stored in the sample at A. The additional energy required to deform the rock sample from A to B is represented by area (3). If the testing machine has sufficient stiffness it is possible to trace out the complete stress-strain curve for Class I rock samples (Vogler, 2014).

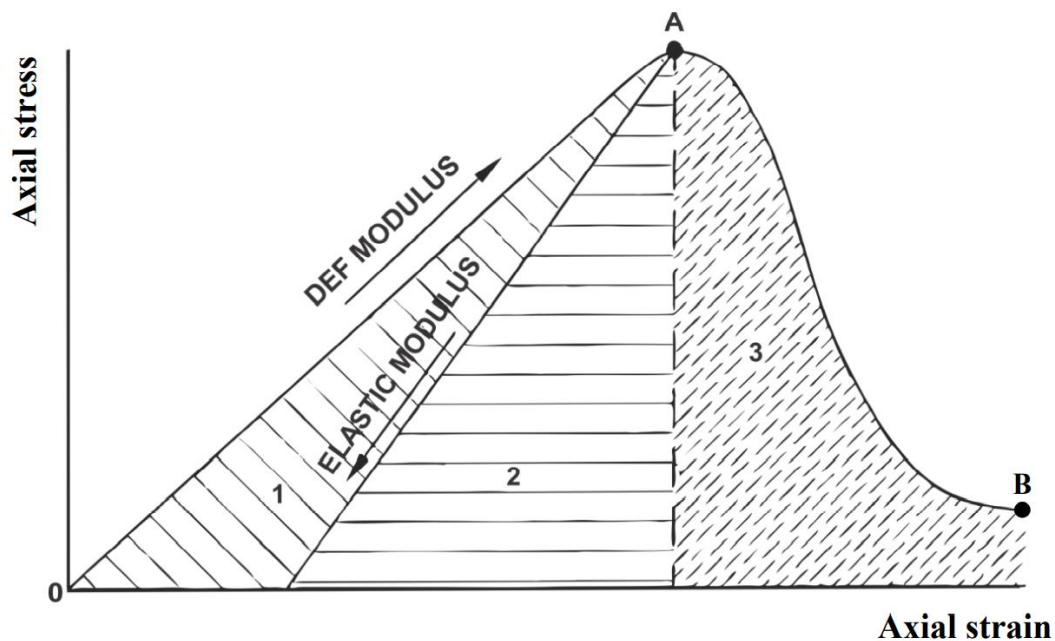


Figure 2.1 Typical Class 1 behaviour (Vogler, 2014).

2.1.4 Class II post-peak behaviour

Class II behaviour is characterized by unstable and self-sustaining fracture propagation. To obtain a complete stress/strain curve, energy must be extracted from the sample being tested. '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' (Vogler, 2014). In other words, machine stiffness on its own

is not adequate to control failure of a Class II rock type. Lateral strain is used as the control (in the post-peak phase) for Class II rock types, as it is the only variable that monotonically increases throughout the rock test (Oniyide, 2015). Figure 2.2 shows a typical Class II stress/strain curve with shaded regions indicating energy supplied by the testing machine and energy required to deform the rock specimen. From Figure 2.2, the axial strain increases from point O to A, then decreases from point A to B after which it starts to increase until point C. Area (1+2+4) represents the total energy supplied by the machine to deform the rock specimen. The energy supplied by the machine to reach peak strength is represented by area (1+2+3). Area (3) represents energy that needs to be extracted from a Class II rock sample. Area (1) shows the non-recoverable energy. Area (2+3) shows the elastic energy present in the sample at point A. The energy required to complete the test from peak strength (from point A to C) is represented by area (2+4) (Vogler, 2014).

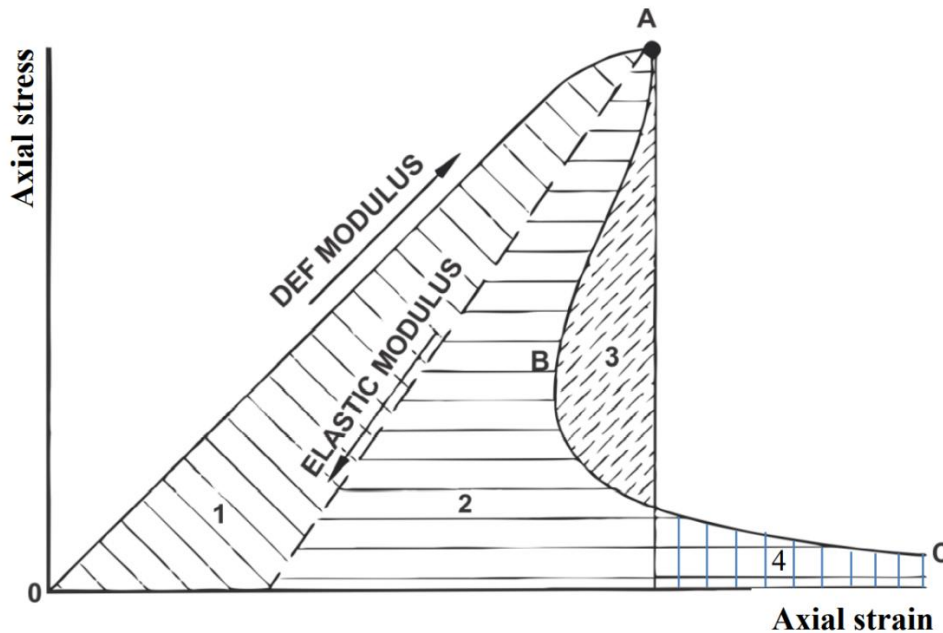


Figure 2.2 Typical class II behaviour (Vogler, 2014)

2.1.5 Servo-controlled testing machine

The main purpose of a servo-controlled testing machine is to trace out the stress-strain curve beyond the peak stress, using the strain in the rock as the controlled variable (Jaeger et al., 2007). In this type of testing machine, deformation is monitored and compared to the desired strain. Any difference between the desired strain and the measured strain creates a correctional signal which adjusts the piston position to bring the strain close to the desired value (Jaeger et al., 2007). Servo-controlled testing machines have very rapid response time (in the range of a few milliseconds). This is quick enough to stop rapid disintegration of the specimen. The machine can trace out the complete stress-strain curve if the lateral strain increases monotonically (Oniyide, 2015). Figure 2.3 shows the closed loop feedback system.

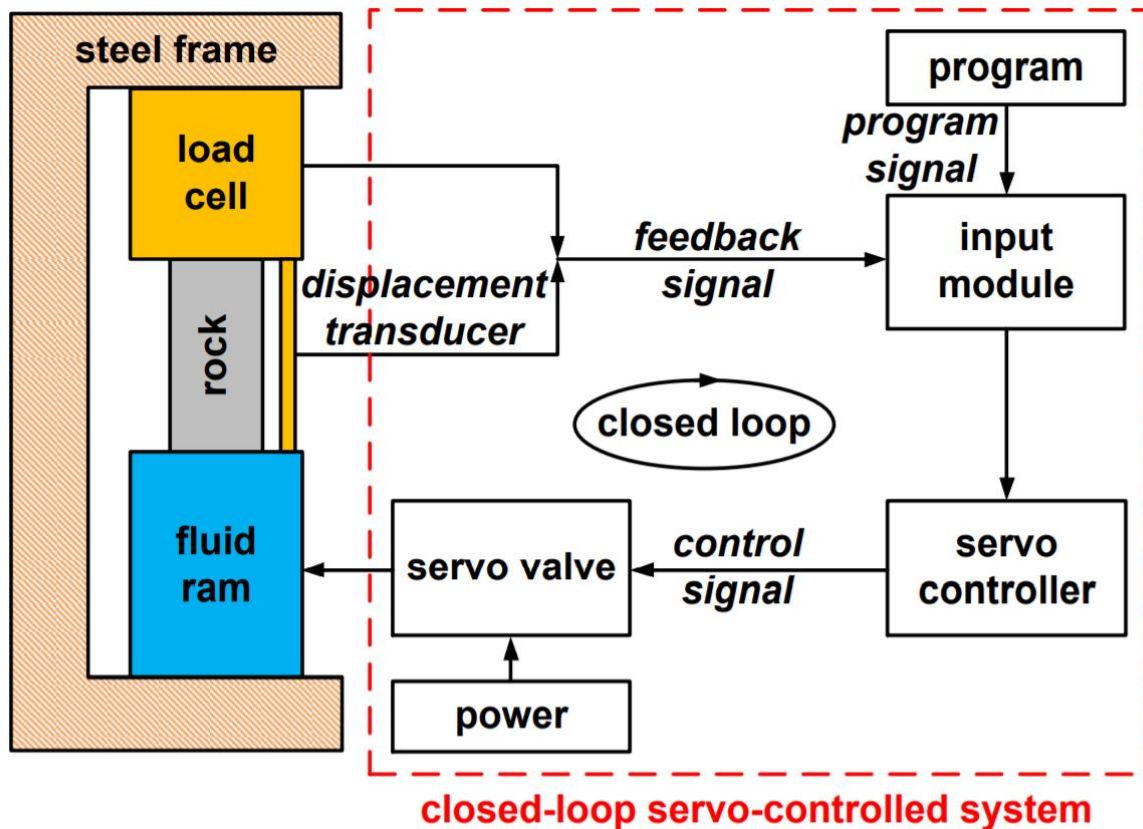


Figure 2.3 Closed servo-controlled feedback loop (Xu, 2017).

Controlled failure can be achieved if the response time of the controlling transducer is quicker than the initiation of rock failure process (Xu, 2017). This allows rock instability to be detected in advance. A servo-controlled pump can be activated by the onset of instability to reduce the fluid pressure rapidly, which in turn increases the effective unloading stiffness of the test machine. Servo-controlled testing machine are designed to ensure slow contraction rate of the rock which allows servo-pump response to be quick enough (Xu, 2017). This ensures that the effective unloading stiffness of the servo-controlled test machine is greater than the post-peak stiffness of the tested rock (Xu, 2017) and the testing machine does not transfer energy to rock sample (which would result in violent failure) allowing the sample to fail in a controlled manner. A servo-controlled testing

machine can be used to trace out the complete stress-strain curve for rock samples exhibiting both Class I and Class II post-peak behaviour.

2.2 Rock Dilatancy

The concept of dilatancy was first introduced by Hansen (1958) to soil mechanics. The dilation angle (ψ) is a parameter that defines the degree to which a shear band increases in volume during slip (Walton and Diederichs, 2015). Under triaxial conditions the volumetric strain is equal to the sum of the axial strain and two times the radial strain as shown in Equation (2-2). The volumetric strain (ε_v) is defined as a measure of the change in volume per unit volume of the material (Vermeer and De Borst, 1984). Vermeer and de Borst (1984) developed an Equation (2-3) to calculate the dilation angle (ψ). This equation holds true for plain strain conditions and triaxial compression. Figure 2.4 shows the idealized bilinear curve.

$$\varepsilon_v = \varepsilon_a + 2\varepsilon_r \quad (2-2)$$

Where ε_v is the volumetric strain, ε_a is the axial strain and ε_r is the radial strain.

$$\sin(\psi) = \frac{\dot{\varepsilon}_v^p}{-2\dot{\varepsilon}_1^p + \dot{\varepsilon}_v^p} \quad (2-3)$$

Where $\dot{\varepsilon}_v^p$ is the volumetric strain and $\dot{\varepsilon}_1^p$ is the principal strain. The superscript p stands for plastic.

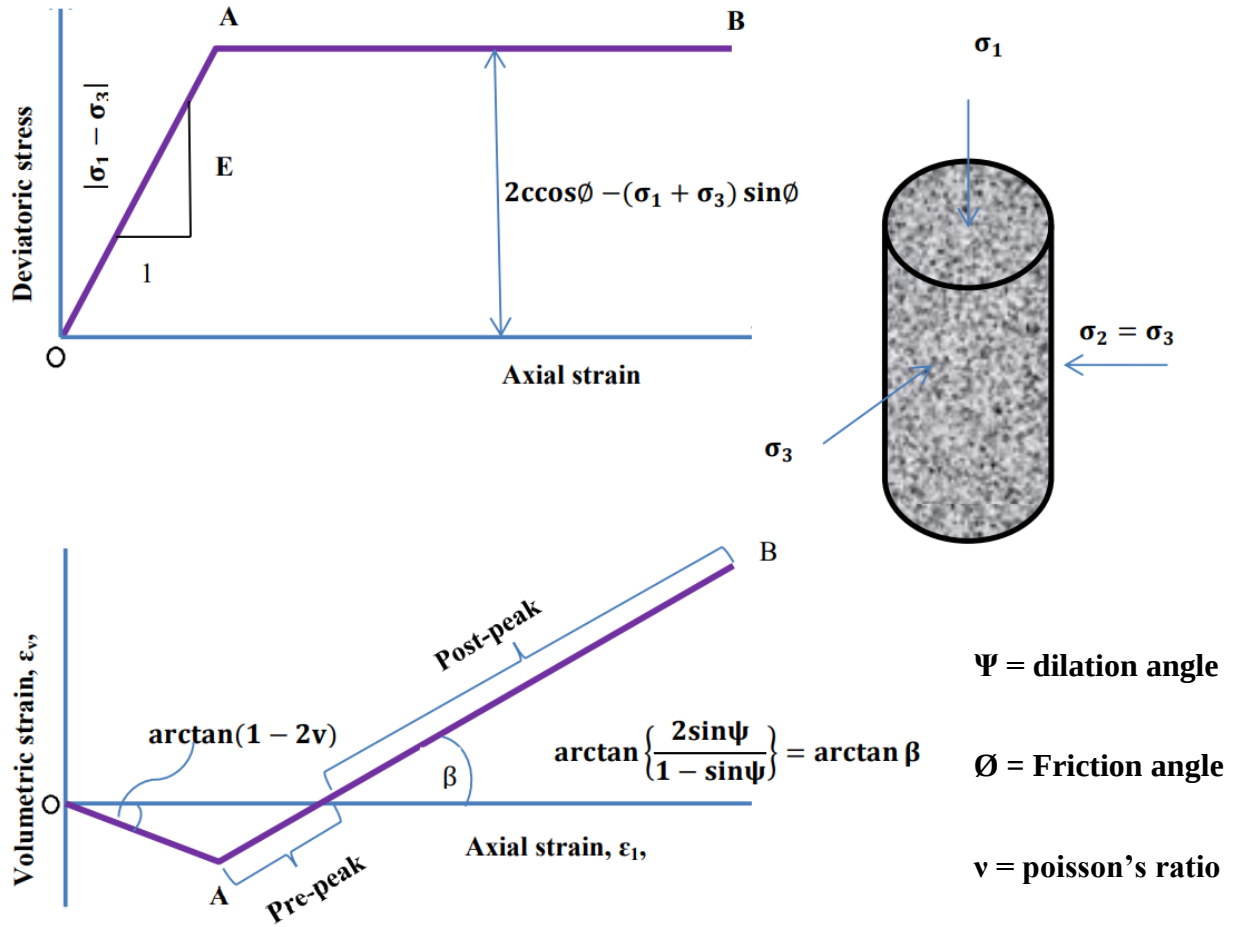


Figure 2.4 Idealized bilinear curve for the determination of the dilation angle. Adapted from (Vermeer and De Borst, 1984).

The initial slope of the idealized curve corresponds to the elastic region (OA). The slope of AB is used in the calculation of ψ and Equation (2-4) is used for this purpose.

$$\psi = \sin^{-1} \left(\frac{\tan \beta}{-2 + \tan \beta} \right) \quad (2-4)$$

Equation 2-5 can be re-written as follows

$$\psi = \sin^{-1} \left(\frac{m}{-2 + m} \right) \quad (2-5)$$

Where m is the gradient of the idealized post-peak curve in Figure 2.4 ($\tan\beta$). Equation 2.5 was used in this research to determine the dilation angle. The dilation angle is sensitive to confinement and tends to decrease with increasing confinement (Walton and Diederichs, 2015). Also, as the confinement increases the onset of dilation is delayed. The angle also varies depending on what stage the rock specimen is at during triaxial testing. The dilation angles in the elastic region are very small or non-existent and are considered insignificant. This is because in the elastic region there is hardly any microcracking taking place in the rock specimen (Vermeer and De Borst, 1984). In the strain hardening regime the rock specimen progressively becomes inelastic because of micro fracturing. The dilation angles in this stage tend to rapidly increase until peak strength. In the strain-softening regime the strain recorded is purely plastic and this is the same when the specimen reaches residual state. At this stage the dilation angles are constant. The dilation angles in the strain -softening regime gradually decrease until the specimen reaches residual state (Walton and Diederichs, 2015).

2.3 Defining failure

In geomechanics, several definitions of failure are described. The first definition describes a state where the rock can no longer support the forces applied to it or otherwise fulfil its engineering functions (Galvin, 2014). A second definition is simply a state where the peak strength is exceeded. The latter definition is used in this research.

2.4 Failure modes in pillars

According to Mathey and Van der Merwe, (2016) there are two failure modes observed in pillars. These are brittle failure and quasi-ductile failure. Brittle failure has been observed in South African mines for pillars with a w/h of less than or equal to 4 (Mathey and van der Merwe, 2016). Brittle pillars exhibit a distinct peak load and if further strain is imposed on the pillar, the strength of the pillar drops to a residual level. The load bearing capacity of brittle pillars is mainly controlled by the cohesive strength of the material. This brittle behaviour is exhibited by crush pillars.

In quasi-ductile failure, the pillar may lose its cohesion entirely, but it maintains or even increases its load-bearing capacity with increasing strain (Mathey and van der Merwe, 2016). This failure mode has been observed in pillars with sufficiently high w/h, which allows enough confinement to be generated in the core of the pillar. Ductile pillars get their seemingly unlimited strength from the shear resistance of the fractured material (Mathey and van der Merwe, 2016).

2.5 Common pillar types found in the Bushveld complex.

2.5.1 Non-yield pillars

These are pillars that provide support to the overburden. This type of pillar is designed to remain intact and elastic throughout the life of mine (Ozbay et al., 1995). They are commonly used in shallow mines. They usually have a width to height ratio (w/h) of 2-5. The pillar size increases as the depth increases. This makes them less economical to use at deep level. The pillar behaviour is shown in Figure 2.5.

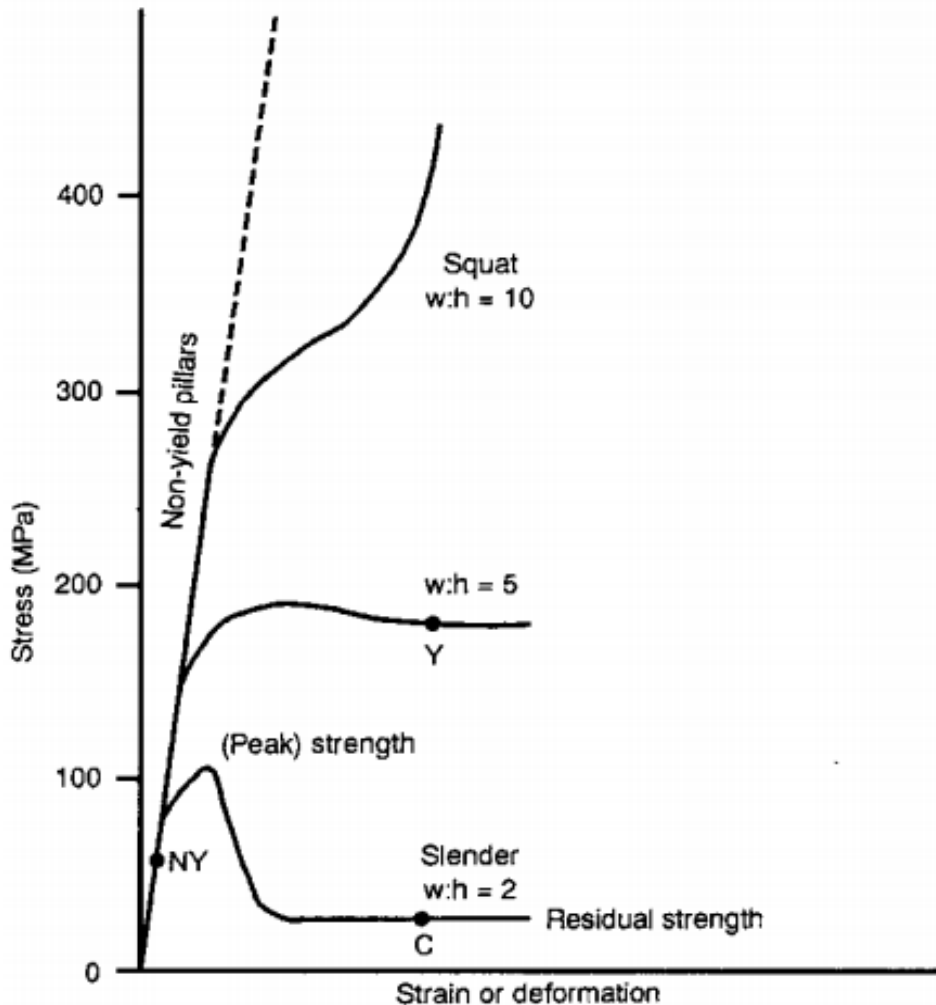


Figure 2.5 Typical stress-strain behaviour of pillars with different width to height ratio. Points Y and C indicate the operating points of yield pillars and crush pillars, respectively.

2.5.2 Barrier pillars

Barrier pillars provide regional support. They have a w/h greater than 5. They are usually oriented towards strike or dip. These pillars are designed to remain intact and elastic throughout the life of mine. They also serve the purpose of compartmentalizing the mine into sections to prevent regional failure of the mine. They also help reduce stope closure and prevent surface subsidence. Barrier pillars also increase the overall strata stiffness

which helps prevent large scale failure (Ozbay et al., 1995). Geological losses are also incorporated into barrier pillars.

2.5.3 Yield pillars

Yield pillars usually have a safety factor greater than or equal to one. They are designed to operate at stress levels close to their peak strength. Yield pillars can only function provided that the surrounding strata stiffness is higher than the absolute value of the post-peak stiffness of the pillar, otherwise violent failure will occur. This type of pillar is commonly used in the transition zone (300m-600m), where crush pillars cannot readily be used (Ozbay et al., 1995). Yield pillars are prone to bursting. Figure 2.5 shows the operating point of yield pillars.

2.5.4 Crush pillars

Crush pillars are designed to support the immediate hangingwall. They are slender pillars with a w/h ratio less than 3 (Watson, 2010). They are usually introduced at depths below 600m. However, they can be used at depths shallower than 600m with caution (Ozbay et al., 1995). At shallow depths, crush pillars become more susceptible to violent failure due to the available stress being lower than the peak strength. This type of pillar is designed to fail close to the face under stiff loading conditions (Ozbay et al., 1995). After failure, the pillar retains some strength which is referred to as the residual strength. The residual strength then provides support of the immediate hangingwall up to the highest active parting plane. The main purpose of crush pillars is to prevent back breaks, for this reason it important to ensure that the crush pillar has sufficient residual strength. The operating point of a crush pillar is shown in Figure 2.5.

2.6 History of crush pillars

To understand how crush pillars operate it is important to look at how they are designed. Crush pillars were first introduced by Korf at Union Section in 1978 to halt stope collapses (Korf, 1978). These pillars had dimensions of 1.5 m by 3 m and a height of about 1 m. Major challenges of stope collapse were being faced when mining exceeded a span of 30 m. Introduction of these smaller pillars that failed close to the face halted stope collapses in the mining areas where they were used (Watson, 2010). Crush pillars have since been commonly used in the Bushveld as in-stope chain pillars oriented along strike or on dip. The design of crush pillars has, in the past, been based on what has historically worked or on calculations based on the peak strength formulae derived from other hard rock mines (Watson et al., 2010). This resulted in oversized pillars which reduced the extraction ratio, and these oversized pillars were also a potential bursting hazard to personnel working underground.

2.6.1 Design of crush pillars

Crush pillars require stiff loading conditions. This loading environment is provided by the face abutments. Fracturing of the pillar should initiate as the pillar is being cut. Observations made by Watson (2010) on the Merensky reef suggest that such loading conditions are present within 5 m to 7 m from the face, and pillars which exceed 10 m without failing would be prone to violent failure. Crush pillars should have reached residual strength by the time loading conditions transition to soft loading in the back area. The residual strength of the pillars provides support to the immediate hangingwall, stabilizing the hangingwall and preventing back breaks (stope collapse) from occurring. The back analysis done by Roberts et al., (2005) at Northam Platinum Mine, showed that a support resistance of about 1 MPa was sufficient to stabilize a stope hangingwall. To meet the

requirement of the 1 MPa criterion, crush pillars must have a residual strength of at least 13 MPa to be able to preclude back breaks (assuming 92% extraction ratio) (Watson, 2010). Oversized crush pillars tend to remain intact close to the face and only fail in the back area where loading conditions are soft and loading stresses are higher. In such conditions the pillars are susceptible to uncontrolled failure.

Different authors have suggested different ways to approach the design of a crush pillar. The most comprehensive methodology for designing crush pillars was provided by Watson, (2010). His methodology points out the three factors that need to be considered when designing a crush pillar.

- Residual strength requirements – this is how much load the failed pillar must be able to support. If the residual strength provided by the pillar is inadequate, back breaks will occur.
- Loading stiffness of the environment, and how this varies with distance from the face- this describes the available stress and stiffness close to the face. For controlled failure to occur, the loading strata stiffness must be greater than the post-peak stiffness (most negative value) of the pillar. The loading stiffness varies with distance from the face. It has been shown that between 5 m to 7 m from the face on the Merensky reef the conditions are stiff enough. The available stress (stress imposed on the pillar) must exceed the peak strength of the pillar for the pillar to fail.
- The relationship between peak pillar strength and w/h ratio (Watson et al., 2010). FLAC models constructed by Watson, (2010) where fracturing was restricted to the pillar (infinitely strong foundation) and where fracturing was allowed to extend into the foundations. With increasing width to height ratio, the pillar model with infinitely strong foundations deviates from a pillar where foundation failure is allowed. For a comparatively ductile pillar, this deviation initiated from a w/h of about 1.2 whilst the more brittle model deviated at about a w/h ratio of 2.5. These points where

deviation takes place indicate punching of the pillar core into the foundation. In the models that allow punching of the foundation, at the respective w/h ratio (1.2 and 2.5) where deviation takes place, the centre of the pillar has become stronger than the foundation inducing fracturing in the foundation materials. In the models with infinitely strong foundations, the pillar strength rapidly increases and the pillars quickly become indestructible. Even when failure is allowed into the foundations, pillar strength increases with w/h ratio. This implies that larger pillars are more likely to remain intact close to the face than smaller ones. To avoid intact or partially failed crush pillars in the back area, the available stress must always exceed the peak pillar strength within 5 m of the face (Watson, 2010). It is important to note that the model assumed that the material that makes up the foundations is the same as the pillars, which is approximately true for the Merensky Reef at Impala Platinum. Figure 2.6 shows a series of FLAC models illustrating the effects of w/h for pillars (Watson, 2010).

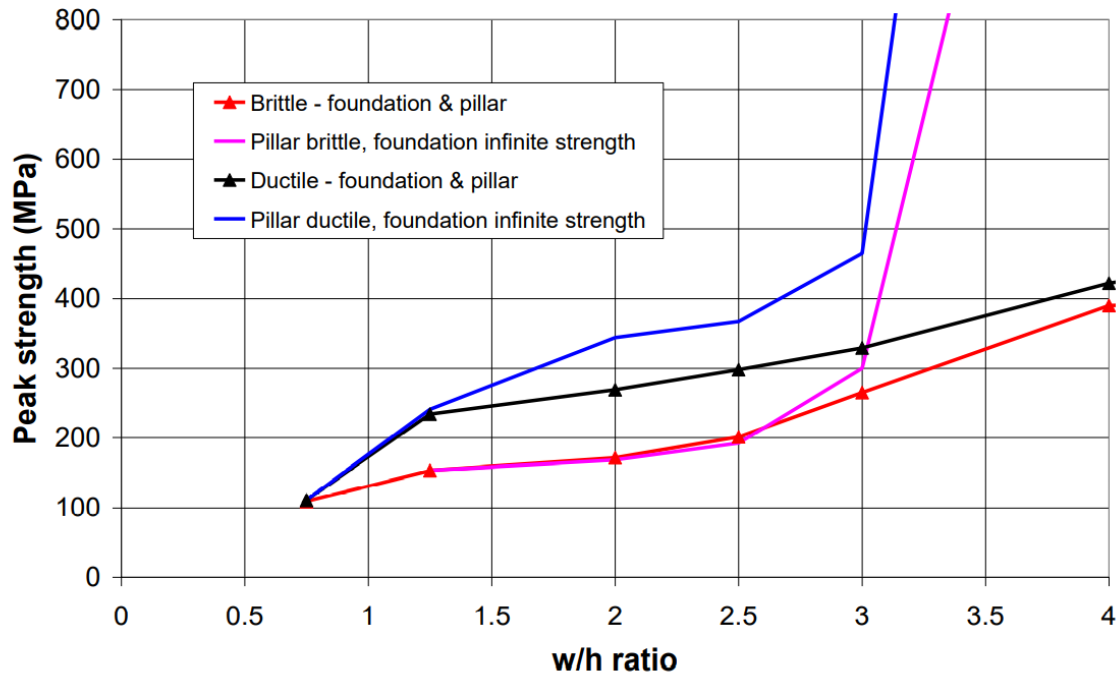


Figure 2.6 FLAC model showing effect of width to height ratio for pillars allowed to punch, as well as for pillars surrounded by infinitely strong rock mass (Watson, 2010).

2.6.2 Typical crush pillar behaviour and challenges associated with crush pillars.

Roberts et al. (2005) estimated that a crush pillar reaches its peak strength at 3-10 millistrain. This is then followed by a strain softening regime caused by further compression of about 5 millistrain to reach residual strength. Work done by Watson (2010) shows that the strength drop occurs along an estimated negative post-peak stiffness slope of 12 GN/m. Further compression of the order of 50-90 millistrain would result in footwall heave which is assumed to be caused by the lateral confinement. At this stage, the foundation strength controls the maximum bearing capacity of the pillar (Watson, 2010).

Crush pillars like any pillar system have challenges of their own. They require good drilling and blasting practice to ensure they are cut to the correct dimensions otherwise they can

become a source of seismic events if they are oversize (Watson et al., 2010). The use of crush pillars results in high stope closure rates, as the pillar continues to deform when it reaches its residual state.

2.7 Effect of mining depth on crush pillars

The depth of mining to a greater extent controls the type of in-stope pillars used. It also influences the mining method used. At shallow depth (less than 300 m), non-yield pillars are commonly used as the in-stope pillars that provide support to the overburden up to the surface (Jager and Ryder, 1999). The use of non-yield pillars becomes less economically viable with increasing depth as the extraction ratio reduces because of the heavier loads that need to be supported. At intermediate mining depths (greater than 600 m but less than 1000 m) it is recommended to use barrier pillars to reduce regional spans and the height of the tensile zone (Jager and Ryder, 1999). This reduces the support resistance required between barrier pillars, as only the immediate hangingwall up to the highest potential parting requires support. Crush pillars may therefore be used between the barrier pillars, as the residual strength of the pillar provides the required support. Crush pillars have the benefit of improving extraction and do not have a potential of instability as the pillar has already failed and reached its residual strength. On the downside, crush pillars allow high stope closure rates to occur and this needs to be taken into account when designing the panel support (Jager and Ryder, 1999). Figure 2.7 shows a typical barrier-crush pillar layout (Watson et al., 2010).

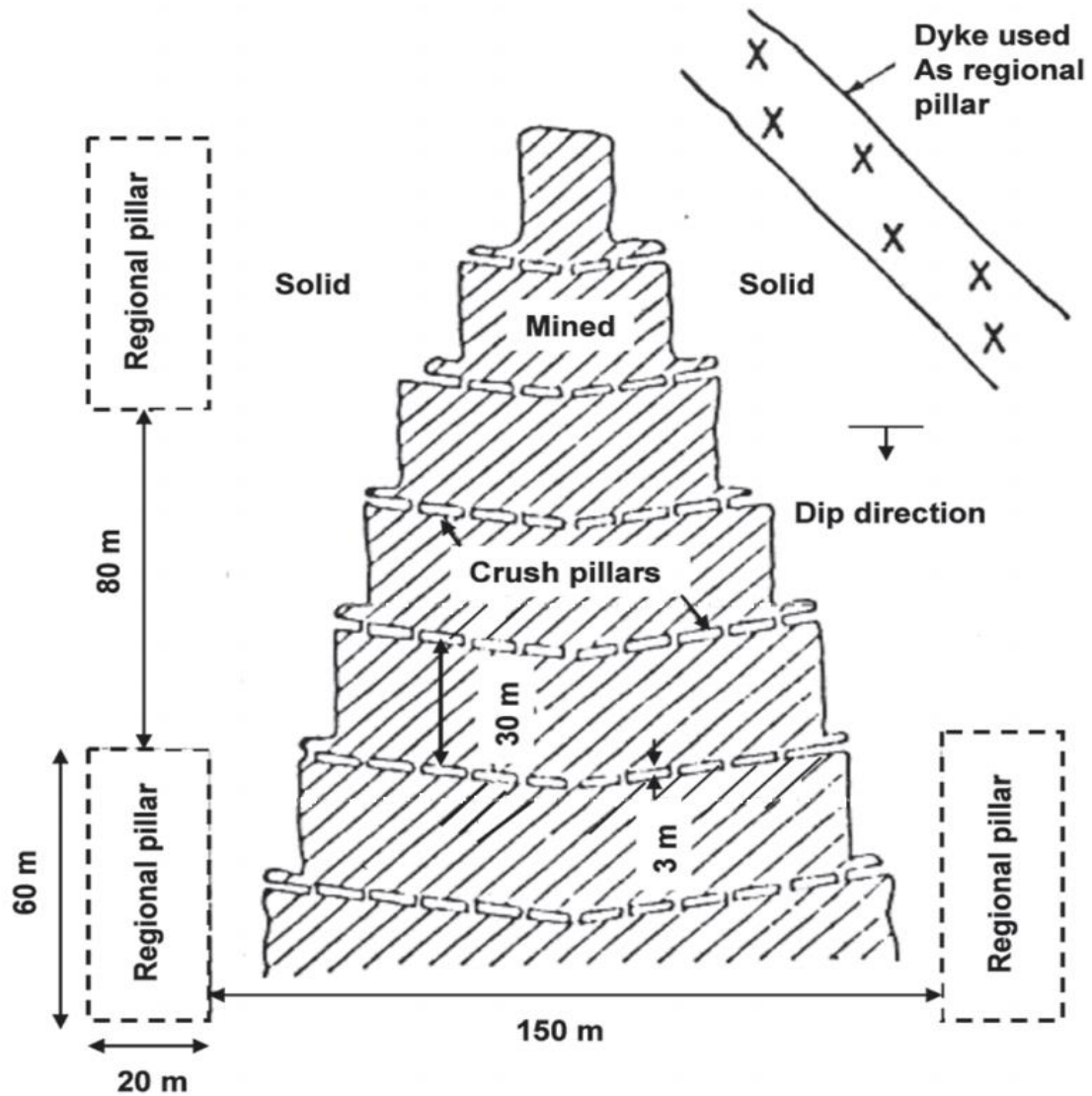


Figure 2.7 Typical barrier-crush pillar layout (Watson et al., 2010).

2.7.1 Transition zone

The transition zone refers to the range of depth (300 m – 600 m) where the pillar stresses are too high for non-yield pillars of reasonable size but too low to initiate fracturing of the crush pillar close to the face (Ozbay et al., 1995). This makes the actual depth to introduce crush pillars difficult to determine. The major challenge with using crush pillars in the transition zone is that failure of the pillar may not occur close to the face, only failing in the

back area where loading conditions are soft and the pillar is prone to violent failure (Du Plessis, 2015). Use of crush pillars in the transition zone is possible but caution is advised. Introduction of smaller pillars may be a solution however, they may not have adequate residual strength to arrest back breaks (Watson, 2010). Use of wider spans can help in increasing the available loading capacity. An alternative approach would be to use small pillars close to the barrier pillars when spans are below a certain dimension, and then increase the size of the pillars when the spans are large enough to fail them at the face. The use of yield pillars has been suggested for the transition zone. Yield pillars are not supposed to lose load as they are designed to operate close to their peak strength where they exhibit strain hardening. However, in practice yield pillars are prone to bursting and are associated with footwall punching resulting in excessive footwall heave (Jager and Ryder, 1999). Jager and Ryder (1999) suggested that it is better to use non-yield pillars up to the depth that crush pillars can be safely introduced. Figure 2.8 shows recommended mining depths for different types of in stope support pillars.

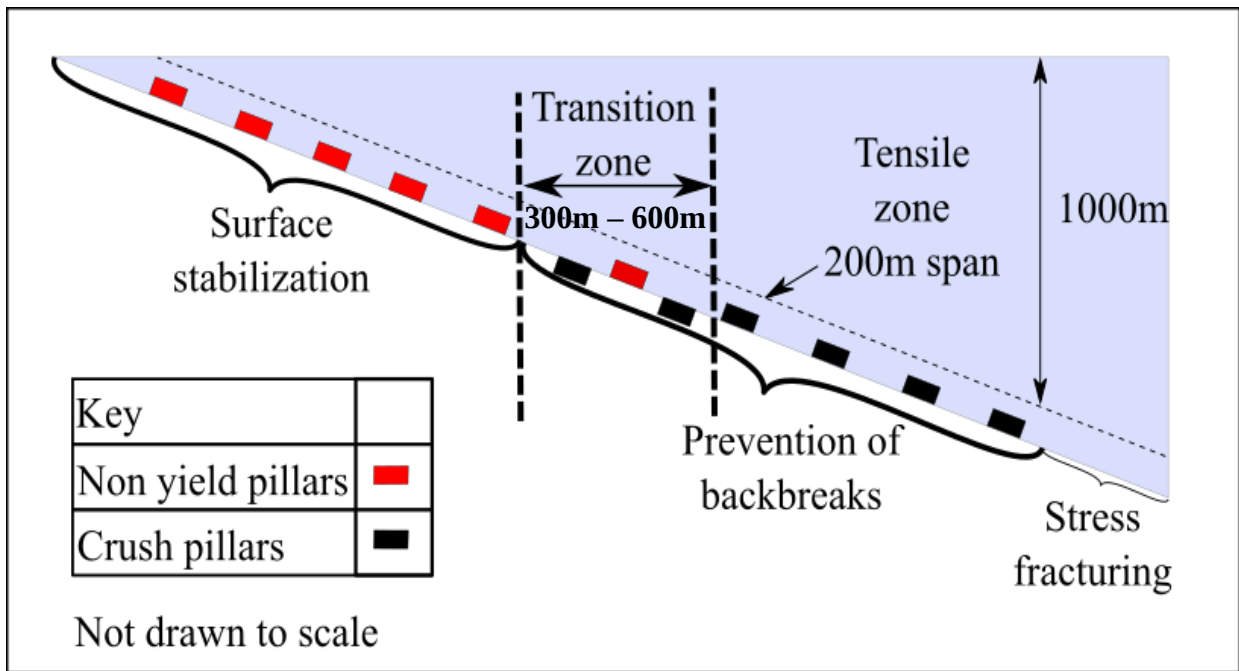


Figure 2.8 Schematic showing the in-stope pillars appropriate for different mining depths (Jager and Ryder, 1999)

2.8 Peak strength

Peak strength is the maximum load that a pillar can support before failing. In the case of a crush pillar, the peak load needs to be exceeded whilst the stiff loading conditions are still present. This makes it important to make precise estimates of the pillar strength.

After the tragic events at Coalbrook Colliery in 1960, Salamon and Munro, (1967) developed the pillar strength formula for coal pillars. The formula estimates the strength of coal pillars with w/h in the range of 0.9 to 3.6 (Salamon and Munro, 1967). The formula underestimates the pillar strength when the w/h is greater than 5 or 6. It is important to note that the formula does not estimate the actual pillar strength but rather it predicts the average strength of a specific group of coal pillars, i.e., those that have collapsed (Mathey and van der Merwe, 2016).

$$Pillar\ strength = Kh^{\alpha}w^{\beta} \text{ (MPa)} \quad (2-6)$$

Where:

$$K = 7.2 \text{ MPa} \quad \alpha = -0.66 \text{ (coal)}$$

$$h = \text{Pillar height (m)} \quad \beta = 0.46 \text{ (coal)}$$

$$w = \text{Pillar width (m)}$$

Hedley and Grant, (1972) adapted the formula for hard rock. A back-analysis of hard rock pillars from the Elliot Lake district in Canada was used to modify the formula (Hedley and Grant, 1972). The pillar strength formula predicted the strength of slender pillars of w/h in the range of 0.7 to 1.5 (Watson, 2010). The modified formula is commonly used by rock engineers in the platinum mines. However, the formula underestimates the pillar strength and does not consider the influence of the foundation. Also, the back analysis was based on only three failed pillars and by Hedley and Grant's own admission, the measurements taken were not accurate and could vary by as much as 50%. For hard rock, the constants were found to be:

$$K = 0.67 \times \text{UCS}$$

$$\alpha = -0.75$$

$$\beta = 0.5$$

The coal formula was revised by Madden (1991) to account for all w/h ratios. The revised formula is known commonly referred to as the squat pillar formula (Madden, 1991).

$$PS = K \frac{R_0^c}{V^d} \left\{ \frac{c}{e} \left[\left(\frac{R}{R_0} \right)^e - 1 \right] + 1 \right\} \quad (2-7)$$

Where K = Rock mass strength

e = Rate of strength increase

V = Pillar volume

c, d = Constants

R = Pillar w/h ratio

R₀ = Critical w/h ratio

Ryder and Ozbay, (1990) suggested a methodology for estimating pillar strength that involves the use of correction factors (F₁ – F₄). These correction factors consider the condition of the pillar, effects of the shape and the effect of w/h ratio. The suggested approach does not consider the effects of the foundation. Pillars with w/h ratio greater than 10 have been observed to withstand any practical load making them essentially indestructible (Mathey and van der Merwe, 2016). However, the pillar system's ultimate strength can still be affected by foundation failure.

According to Watson (2010), Bieniawski and Van Heerden (1975) (cited by Watson) suggested that a linear strengthening law gave more reliable pillar strength estimates than the Salamon-Munro power formula.

$$PS = K_i \left[b + (1 - b) \frac{w}{h} \right] \quad (2-8)$$

Where K_i = In situ cube strength

b = constant

W = Pillar width

h = Pillar height

Based on research at Impala Mine, Watson developed a linear formula using the maximum likelihood back analysis to estimate pillar strength on the Merensky reef (Watson et al., 2008). This formula is very similar to Bieniawski and Van Heerden (1975) with the only difference being that Watson's formula uses the effective width and height.

$$Strength = K \left[b + (1 - b) \frac{w_e}{h_e} \right] \quad (2-9)$$

Where $h_e \approx \left[1 + 0.2692 \left(\frac{w}{h} \right)^{0.08} \right] h$

$$w_e \approx 2wL / w + L$$

K = in situ cube strength

$$b = w / h_e$$

w = pillar width

h = pillar height

L = pillar length

A strength formula was developed for UG2 by Watson (2021). The formula parameters are based on the back analysis done on (failed and unfailed) pillars at three platinum mines on the UG2 reef. The formula is based on the power formula with exponents of α and β and the use of effective height and width. The effective width considers the presence of gullies unprotected by sidings. The adjustment is based on numerical modelling done by Roberts et al. (2002). Equation 2-10 (Watson et al., 2021).

$$Pillar\ strength = Kw_e^\alpha h_e^\beta \quad (2-10)$$

Where $h_e \approx \left[1 + 0.2692 \left(\frac{w}{h_g} \right)^{0.08} \right]$

W_e = effective width

h_g = gully vertical height

$\alpha = 0.67$

$\beta = -0.32$

$K = 67$

The modified power formula applicability to the UG2 reef will be tested and compared with the results from modelling. Peak strength of a pillar can be estimated using a numerical model. Numerical models can handle complex geometries. However, the models need to be calibrated using either underground measurements or data from laboratory testing. In this research Numerical modelling is used to estimate the peak strength of a crush pillar.

2.8.1 Effect of width to height ratio on pillar strength

Tests done by Bieniawski (1968) on sandstone showed a linear increase in peak strength as the w/h increased, up to a w/h of 5. Samples tested at higher w/h ratios could not be broken even at higher loads (Mathey and van der Merwe, 2016, citing Bieniawski). Results presented by Bieniawski and Van Heerden (1975) showed a similar trend of linear strength increase with increasing w/h ratio up to a w/h ratio of 4. Model pillars with w/h in the range 5-7.5 showed an exponential rise in strength, suggesting the brittle-ductile transition had already occurred (Bieniawski and Van Heerden, 1975). Madden (1991) also performed tests on sandstone pillar models and the results show a linear increase in strength up to a w/h of 6, after which a rapid rise in strength was observed. The results obtained from pillar models suggest that for pillars with w/h of less 6 their peak strength can be predicted by a linear equation (Madden, 1991). Numerical models done by Su and Hasenfus (1999) showed that the ultimate pillar strength of pillars with w/h > 10 was dependent on the

competency of the surrounding strata. Weak floor for example could reduce the ultimate pillar strength by as much as 50% (Su and Hasenfus, 1999).

2.8.2 The effects of Foundation draping.

Foundation (hangingwall/footwall) draping causes the edges of the pillar to experience high stresses. This causes pillar failure to initiate from the edges and progress towards the centre (Watson, 2010). The interface between the pillar and the foundation controls how much horizontal stress is transferred to the pillar. The frictional resistance at the interface regulates the horizontal stress transferred to the pillar. The higher the frictional resistance the greater the horizontal stresses that are transferred to the pillar. The transferred horizontal stress provides confinement to the core thereby strengthening it (Watson, 2010). The influence of Foundation draping is controlled by the mining spans around the pillar (Watson, 2010). The greater the mining spans around the pillar, the more prevalent the effects of foundation draping become. The effect of foundation draping should be considered when simulating pillar behaviour. Figure 2.9 shows an illustration of foundation draping (Watson, 2010).

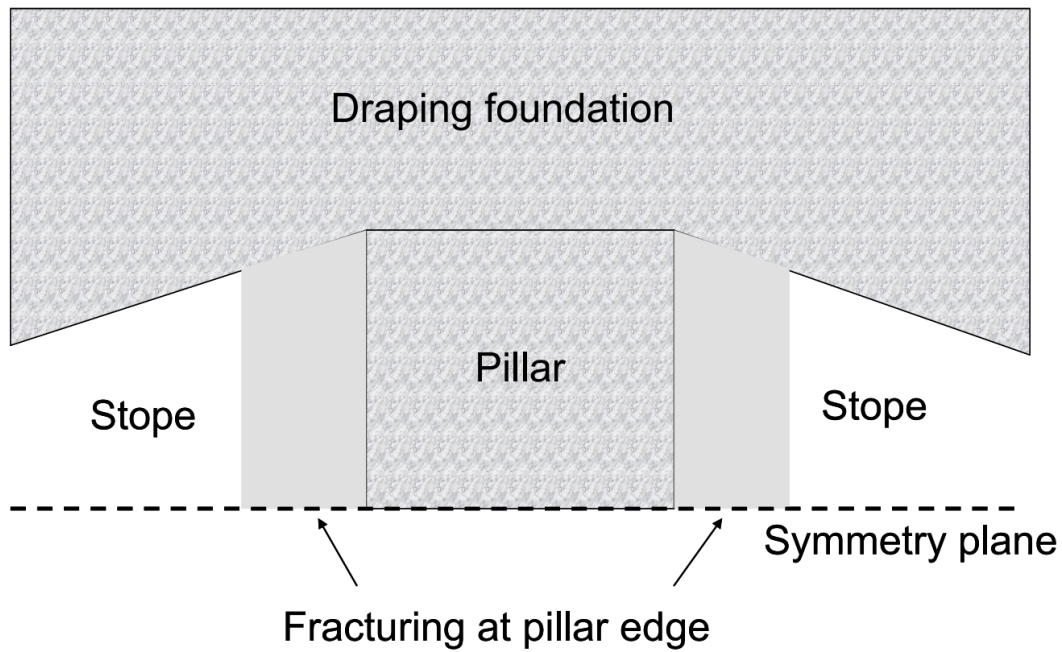


Figure 2.9 Illustration showing foundation draping (Watson, 2010).

2.9 Pillar Foundation

A pillar foundation is a crucial part of the pillar system. It comprises of the hanging- and footwall. Tests conducted by Lougher and Ozbay (1995) showed that pillar stress for 6 m × 3 m pillars ($w/h = 3$) increased with the extent of mining up to an average pillar stress of 208 MPa. Further mining resulted in the punching of the footwall and the stress level remained approximately the same (Lougher and Ozbay, 1995). This suggests that the foundation may have great influence on pillar failure.

2.9.1 Pillar system failure

A pillar system comprises of the pillar, the adjacent footwall and hangingwall. According to Watson, (2010) pillar system failure is a combination of four mechanisms. The mechanisms are listed as follows:

- Fracturing and crushing of the pillar;
- Tensile fracturing into the foundation (Hertzian crack) (Watson et al., 2010);
- Wedge formation in the form of shear fractures (Prandtl, 1921); and
- Horizontal dilation of wedges. (Control the ultimate punch resistance).

Fracturing and crushing of the pillar starts in the early stages of pillar failure and continues even when other mechanisms become active. This is usually followed by the fracturing of the foundation. Tensile stresses are induced in the foundation as the load exerted by the pillar to the foundation increases (Watson, 2010). This results in the formation of tensile fractures in the proximity of the pillar. These fractures are called the Hertzian cracks. The development of shear fractures marks the beginning of wedge formation. Modelling done by Watson (2010), showed extensive foundation failure with large solid wedges at the core of the pillar. Watson identified these wedges as the Prandtl wedge (Prandtl, 1921). The Prandtl wedge is a shear failure mechanism in the foundation below a highly stressed stability pillar (Baars, 2015). The unique characteristics of this wedge are that it consists of three zones as shown in figure 2.10. Zone 1 is a triangular zone underneath the pillar which is under elastic equilibrium. The absence of friction (between the strip load and zone 1) implies that the direction of the principal stresses are vertical and horizontal, with the largest principal stress in the vertical direction (Baars, 2015). Zone 2 is in a state of plastic (inelastic) equilibrium and has the shape of logarithmic spirals. The principal stresses are rotated 90° from zone 1 to zone 3. Zone 3 is a passive zone of shear failure. It should be noted that the mentioned zones are applicable to soils (Baars, 2015). No evidence was found in literature that show that these zones are applicable to hard rock.

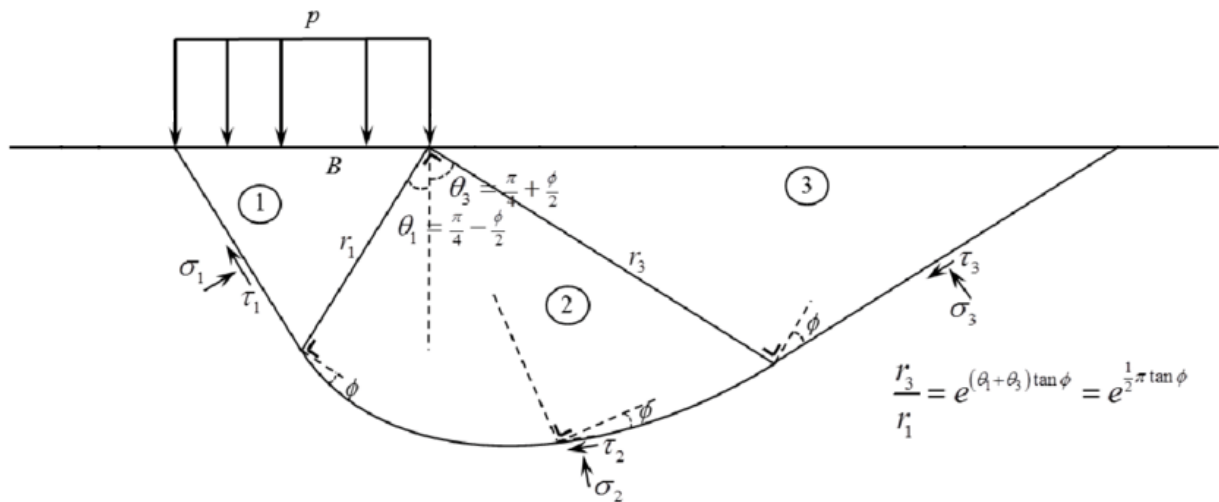


Figure 2.10 The Prandtl wedge (Baars, 2015)

Although Watson (2010) did not show if the Prandtl wedge in hard rock is also composed of three zones as in soils, his modelling showed they exhibit similar behaviour. The wedge provides a slip line (shear fracture) which enables the footwall (foundation) to dilate into the stope, creating room for pillar deformation and failure. Foundation failure is the final stage of pillar failure. Figure 2.11 shows a simplified diagram of the Prandtl wedge (Watson, 2010).

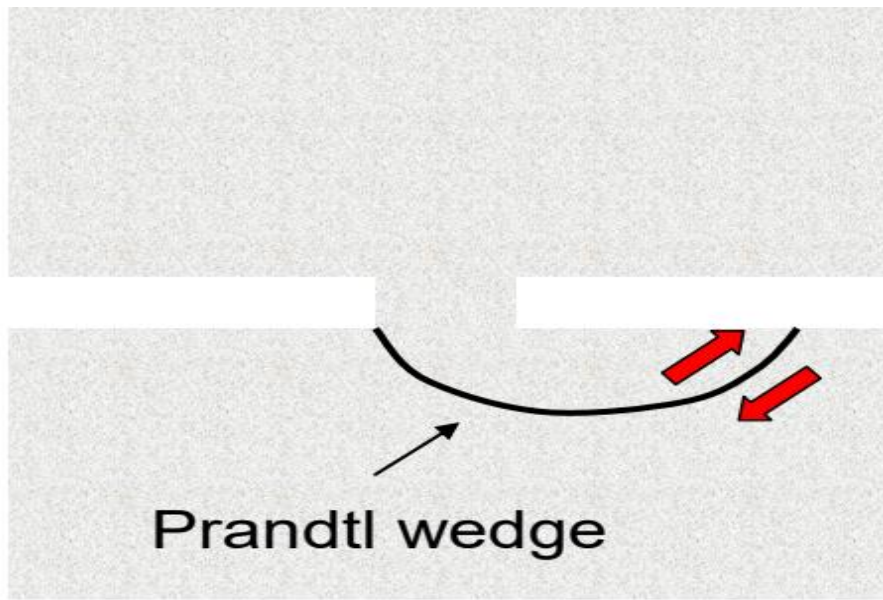


Figure 2.11 Diagram show a typical Prandtl wedge shape (Watson, 2010)

The horizontal resistance against the dilation of wedges controls the ultimate punch resistance. Ryder and Jager, (2002) expressed the ultimate punching resistance as follow:

$$\sigma_p = \frac{1 + \sin \phi}{1 - \sin \phi} UCS_h \quad (2-11)$$

σ_p = Punch strength

ϕ = Internal friction angle

UCS_h = In situ horizontal strength

2.10 Post-peak behaviour of pillars

When the peak strength of a pillar is exceeded, the pillar enters the post-peak regime. Pillar failure can be either stable or unstable. The stability of the failure process is governed by relative stiffness of the pillar and the loading strata. This is similar to laboratory tests where the machine and sample stiffness determine whether or not violent

failure will occur. Salamon (1970), developed the local mine stiffness stability criterion based on observations made on laboratory tests.

2.10.1 Local mine stiffness stability criterion

The local mine stiffness stability criterion developed by Salamon (1970) predicts if a pillar would fail in a controlled manner or result in violent failure.

For stable failure.

For unstable failure

$$|K_{LMS}| > |K_P|$$

$$|K_{LMS}| < |K_P|$$

Where K_{LMS} is the local mine stiffness and K_P is the pillar stiffness (Salamon, 1970). The criterion does not have a time dependant variable, meaning it cannot predict how quick or slow the failure process would be (Zipf, 1999). Slow yielding pillars allow personnel ample time to evacuate unlike a sudden failure. K_{LMS} is affected by the extraction ratio and tends to decrease with increasing extraction ratio (Zipf, 1999). If a small amount of deformation results for mining a certain distance, the system is stiff. For a soft system, a large deformation would occur for the same distance mined.

2.10.2 Strata stiffness

Strata stiffness is defined by Ozbay, (1989) as the force required to cause a unit increment of closure between the hangingwall and footwall at the location of the pillar. It varies with distance between support elements (strata stiffness decreases with increase in span) and is dependent on the surrounding extraction pattern (Ozbay, 1989). The stiffness of the loading strata governs the ability of the hangingwall to redistribute load to surrounding

elements providing support. This was illustrated by Kersten, (2016) who looked at the extreme cases of strata stiffness, i.e. a very stiff and soft hangingwall. In his illustration both cases were supported by equally spaced springs with varying moduli. In the case of extreme hangingwall stiffness, each spring had the same convergence but with different resistance force. With a totally soft hangingwall, each spring supports a strip of the total overburden load resulting in varying convergence for each spring. This corresponds to the concept of the tributary area. Figure 2.12 shows an illustration of the extreme cases of hangingwall stiffness.

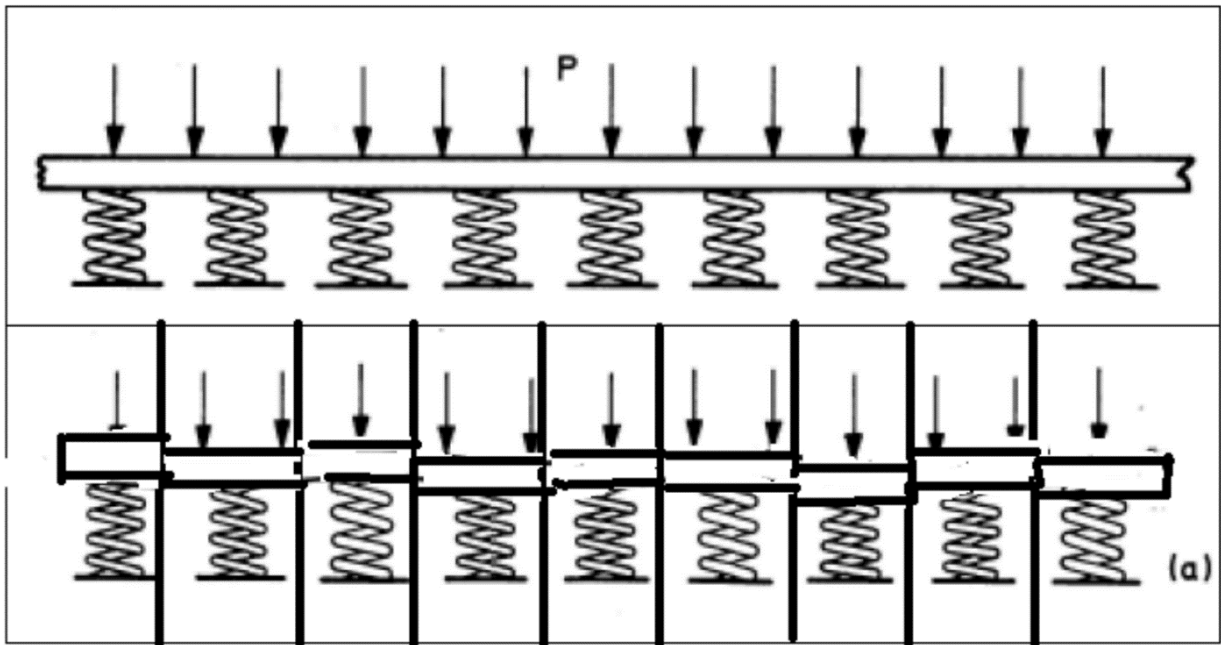


Figure 2.12 Illustration showing extreme cases of hangingwall stiffness. At the top shows the case of totally stiff hangingwall and at the bottom shows case of totally soft hangingwall (Kersten, 2016).

Crush pillars are supposed to start failing as soon as they are cut at the face. Part of the energy supplied by the loading strata to deform the crush pillar is used in the creation of

new fractures and the rest is stored in the system as strain energy, up until the peak strength of the pillar is exceeded (Galvin, 2014). After exceeding the peak strength, the stored energy is available to drive further deformation of the pillar. The softer the loading strata, the greater the energy stored, and the greater the energy available to cause more deformation. If the stored energy exceeds the energy required to cause further deformation and fracturing, the system becomes unstable resulting in violent failure. It is crucial that a crush pillar fails close to the face where loading conditions are stiff enough allowing controlled failure to occur.

2.10.3 Residual strength

The residual strength of a crush pillar serves the fundamental purpose of preventing back breaks. This strength is retained after the pillar fails (Roberts et al., 2005). Some authors attribute this to cohesion between the failed material. Roberts et al. (2005) suggest that crush pillars retain their residual strength from the self-confinement within the failed reef or rock material. A combination of confinement and friction between fragments is a more likely source of residual strength. After yielding, the pillar ends up as an intensely fractured but still well-knit mass of crushed rock (Ozbay et al., 1995). According to Du Plessis, (2015) at depths greater than 600 m, crush pillars reach their residual strength at approximately 9 m from the face.

Several attempts have been made to estimate the residual strength of a crush pillar. Ryder and Ozbay (1990) estimated that the residual strength of a crush pillar was 5% to 10% of the peak strength. This translates to a residual strength between 8 MPa and 20 MPa for UG2. Roberts et al. (2005) conducted 2D stress measurements on the Merensky reef and estimated a residual strength of 19 MPa. Underground measurements done by Watson

(2010) on the Merensky reef pillars at Impala mine showed the pillars had a peak strength of about 320 MPa and residual strength of 32 MPa. In this research the residual strength for UG2 crush pillar will be estimated using numerical modelling and the model will be calibrated using data from literature and laboratory tests.

2.11 Numerical modelling of pillars

All models used in this research were done using FLAC3D. According to Watson et al. (2010), when building a pillar model, the brittleness of the material must be considered. The material brittleness in the model is defined as “the rate of stress decrease after failure” (Watson et al., 2010). Unfortunately, in FLAC3D this parameter is also affected by grid size. The issue is more prevalent in models that involve pillars with large w/h ratios (Watson et al., 2010). In such a case failure progression can reach the core of a pillar before the peak strength is reached. The pillars that were investigated in this research are categorized as slender pillars (having a w/h ratio of less than 2) and the grid size does not have as much influence on the model. As a precaution, the issue was considered in the models used in this research. The friction angle and the dilation angle were kept constant to simplify the model. Only the cohesion was varied. This means the post-peak behaviour (brittleness) was controlled by cohesion loss. This is similar to what was done by Watson et al. (2010).

Boundary conditions are important in pillar modelling and can directly affect the results of the model (Itasca, 2019). Two main types of boundaries that can be used in FLAC3D namely: prescribed displacement and prescribed stress. The models used in this research used prescribed displacement boundaries. Roller and velocity boundaries fall under prescribed displacement boundaries. Roller boundaries are recommended to be used on

planes where conditions are the same on either side (Itasca, 2019). This is usually along planes of symmetry. Velocity boundaries apply a velocity normal to the specified plane. This boundary was used to load the pillar models used in this research.

The models used in this research involve the presence of a gully next to a crush pillar. The estimated strengths from the models and the formula developed by Watson (2021) for UG2 were compared. The formula requires an effective height as an input. The effective height considers the presence of the gully next to the pillar. The equation (2-10) used to calculate the effective height is based on numerical modelling done by Roberts et al. (2002), to account for asymmetric pillars. The presence of a gully next to a pillar changes the effective height of the pillar (Watson, 2010). The models used in this research investigate the effect of a gully on the peak strength of a pillar.

2.12 Chapter summary

From the literature survey, it has been established that stress plays a pivotal role in a crush pillar's performance. The available stress, peak and residual strength must be considered when dealing with crush pillars. The available stress at the face must be greater than the peak strength to ensure pillar failure initiates as the pillar is being cut. It is also important to note that being at depths where crush pillars can be used does not necessarily mean there will be sufficient stress at the face to initiate failure. Pillar and foundation rock properties, presence of weak layers and stiffness (system and pillar) also play a huge role in crush pillar behaviour and design.

3 LABORATORY TESTING

To accomplish the objectives of the research study, it was necessary to test both pre- and post-failure behaviour of rock samples. During the laboratory testing, some problems were encountered. These include oil spoiling samples during testing and inaccurate reading from the axial transducer. The first part of the chapter deals with the problems encountered and the second part with the laboratory test results.

The inaccurate readings from the axial transducer raised questions about the calibration of the instruments used in this research. It is important that all equipment used is correctly calibrated as this affects the results obtained. Without calibration, the results obtained cannot be compared with the works of other researchers as there is no standard on which the results are based. This makes it difficult to draw meaningful conclusions and contribute to the available knowledge.

3.1 Calibration

The MTS 815 servo-controlled testing machine that was used in this research was standing idle for 3 years and required calibration. The machine was calibrated before any rock testing was done, however, instead of calibrating the machine for strain (mm/mm), it was set to measure deformation (mm). This meant the recommended values (by ISRM) for control variables (given in strain) for post-peak investigations had to be converted to deformation using Equation 3-1.

$$\Delta l = \varepsilon l \quad (3-1)$$

Where Δl = Deformation

ε = Strain

l = original length

The original length is the distance between the centres of the mounting points of the transducer as shown in Figure 3.1. The original length for the axial transducer is 50 mm. To calculate the strain from the data collected from the transducers Equations 3-1 and 3-2 were used. To determine the strain from the axial transducer's recorded data, Equation 3-1 was rearranged by making strain subject of the formula. Where Δl is the deformation reading from the axial transducer and l is the original length (50 mm). Equation 3-2 was used to calculate strain from the radial transducer's data.

$$\varepsilon = \frac{\Delta l_d}{\pi D} \quad (3-2)$$

Where Δl_d = Deformation

D = Diameter of specimen

The radial transducer measures circumferential deformation, hence the use of equation 3-2.

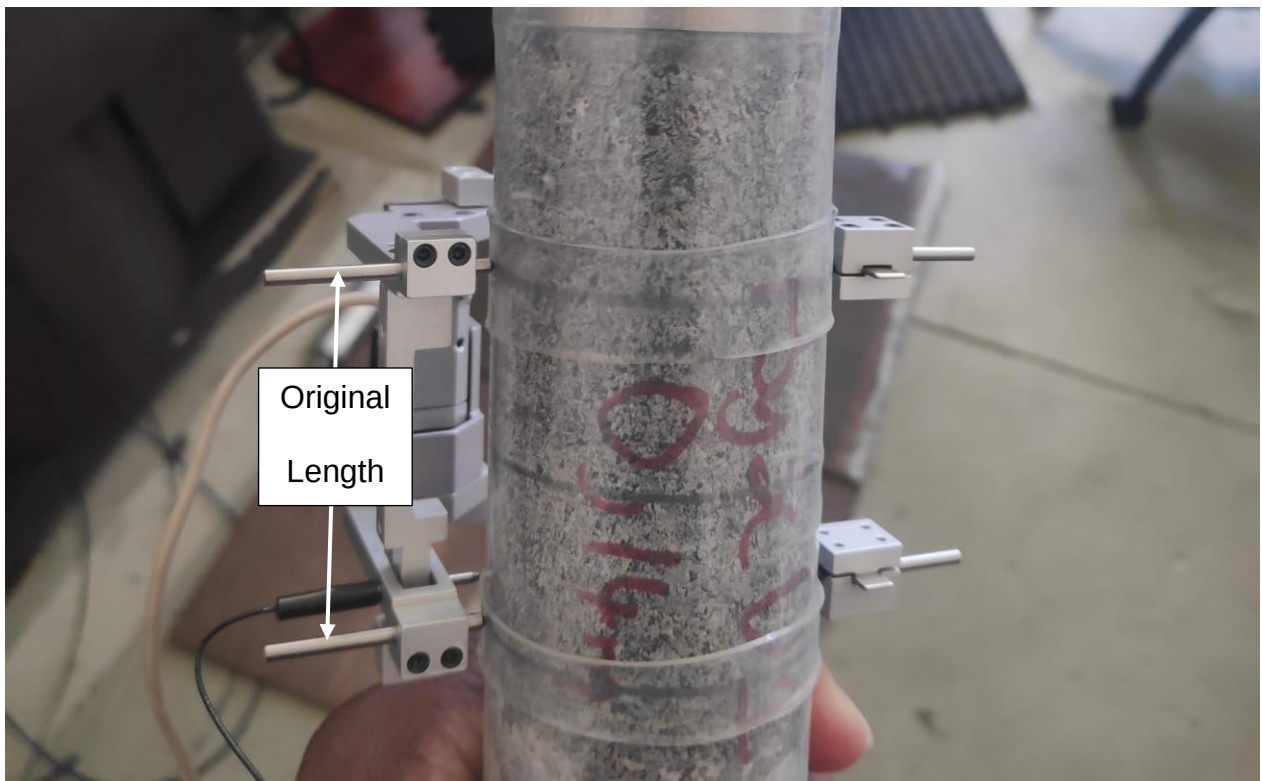


Figure 3.1 The original length of the axial extensometer

3.2 Challenges faced during rock testing

Obtaining a complete stress/strain curve has its own inherent challenges. The challenges discussed in this section are those faced by the author while conducting post-peak rock tests. All rock testing procedures used in this research were based on the ISRM suggested methodologies (Ulusay and Hudson, 2007).

3.2.1 Oil contamination

It took many attempts until the first complete stress/strain curve was obtained. The post-peak triaxial test would either fail prematurely or the system would abruptly stop the test. There were a few contributing factors, but the issue of oil contamination of the specimen was the most prevalent. During triaxial tests, the specimen was protected from oil contamination using a shrink sleeve. There are two different products of heat shrinkable

sleeves available in the rock engineering test laboratory. These are the American imported sleeve (referred to as the American sleeve) and the locally sourced sleeve (referred to as the local sleeve). The American sleeve is hard in comparison to the local sleeve and the contact points of the axial transducers are not able to punch through it. However, there is little stock available, and the sleeve is reserved for performing triaxial tests at high confinements.

The local sleeve has plenty of stock available, but it has the problem of being punched by the contact points of the axial transducer and allowing oil to infiltrate the specimen. This infiltration usually occurred at failure and therefore did not allow post-failure. The problem was resolved by introducing a second layer of the local sleeve at the points of contact of the axial transducer (Figure 3.1). It was also observed that a 12-hour cooling period after applying the local sleeve greatly improved its performance.

3.2.2 Test control procedures

The ISRM procedures for obtaining a complete stress/strain curve of a specimen consist of three stages. The first stage is to use axial strain as the control variable (during pre-failure testing) and at around 70% of the UCS change to the second control variable which is radial strain. The third stage involves changing back to axial strain control when the stress drops to about 50% of the peak strength. The software used to operate the MTS machine does not allow any modification to be made during the test. All instructions are pre-programmed before commencing the test. It proved difficult to determine a changeover point for the third stage and all attempts to predict this point resulted in the machine losing control and the test ending abruptly. It was decided to exclude the third

stage as the specimen could still reach residual strength with only radial strain control. However, this made the duration of the triaxial tests very long (between 8 – 10 hours).

3.2.3 Problems with the Axial transducer

After resolving the issue of oil contamination and modifying the test procedure, the success rate of obtaining a complete stress/strain curve greatly improved. However, the results from the triaxial tests became illogical and not comparable to previous tests. The problem usually initiated close to the peak strength and the stress/strain curve would deviate from the expected result, producing stress/strain curves that cross the pre-failure region (Figure 3.2). The problem worsened with time. The source of the problematic results seemed to be emanating from the axial transducer (extensometer). The problem started off very sporadically, however, it became a prevalent issue as more rock tests were conducted. An example of this issue is shown in Figure 3.2 which shows a stress-strain curve obtained after performing a triaxial test at 30 MPa on an anorthosite sample. The shape of the curve cannot be classified either class I or class II as observed by Wawersik and Fairhurst (1970).

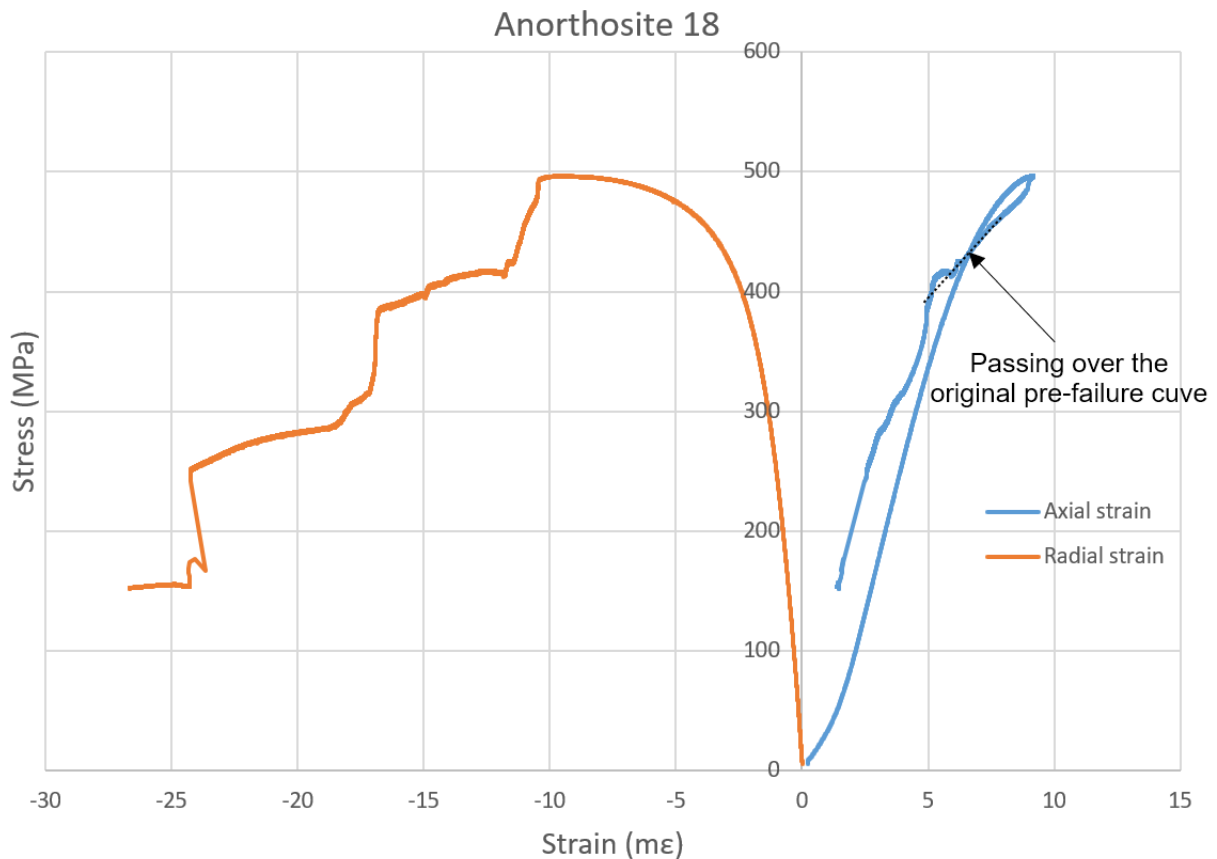


Figure 3.2 Stress/strain curve of anorthosite sample at 30MPa confinement.

The data from the radial extensometer were logical and produced stress/strain curves that were consistent with expectations. The stress/strain curves of specimens of the same rock had similar shapes. This was not the case with the axial transducer. The stress/strain curves produced had unique shapes even for specimens from the same rock type. The problem seemed to initiate as the stress levels approached peak strength or after the peak strength had been exceeded. With no logical explanation for what was causing the axial extensometer to behave in this manner, it was decided to investigate if the transducer was malfunctioning. Tests were conducted to determine the cause.

3.2.4 Investigating transducer issues

A series of tests was developed to confirm if the problem lay with the axial transducer. The first test was to determine the effect of confining pressure on the transducer, secondly cross talking between the two transducers was investigated. Cross talking occurs when the electrical signals from one transducer are being mixed with the other. The extensometers were placed in an empty triaxial cell as shown Figure 3.3, the axial extensometer was on top with the radial extensometer being beneath.

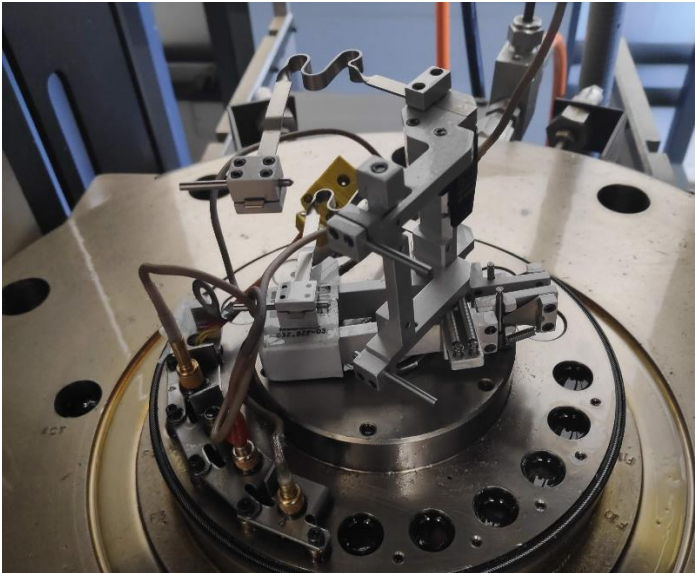


Figure 3.3 Axial (top) and radial extensometer in an empty triaxial cell

The triaxial cell was filled with oil and the confining pressure raised at a rate of 10 MPa per minute until 60 MPa. The confining pressure was held constant at 60 MPa for 5 minutes after which the confining pressure was then reduced back to zero at a rate of 10 MPa per minute. The test was again repeated with each transducer separately. The expected results were supposed to show no change in the deformation readings (constant at zero) throughout the test. The results of the two extensometers show decreasing

deformation readings as the confining pressure increased. As the confining pressure decreased the axial extensometer showed decreasing deformation readings whilst the radial extensometer showed increasing readings. At constant pressure both extensometers showed a slight drop in deformation readings. The results of the radial extensometer showed a repeatable pattern, suggesting the instrument could have a constant error that can be easily adjusted. However, the axial extensometer's deformation readings continuously decreased throughout the test, despite the changes in pressure. At the end of the test the axial deformation reading was -0.074 mm and this exceeded the error range of ± 0.03 . The results suggested that the axial extensometer was malfunctioning. However, this was not conclusive and additional testing was required. There was no evidence of cross talking as the results of the transducers when tested separately produced similar results. The results from the tests are shown in Figure 3.4. The results suggest that these types of extensometers are not suitable to be used in conditions with varying confining pressure.

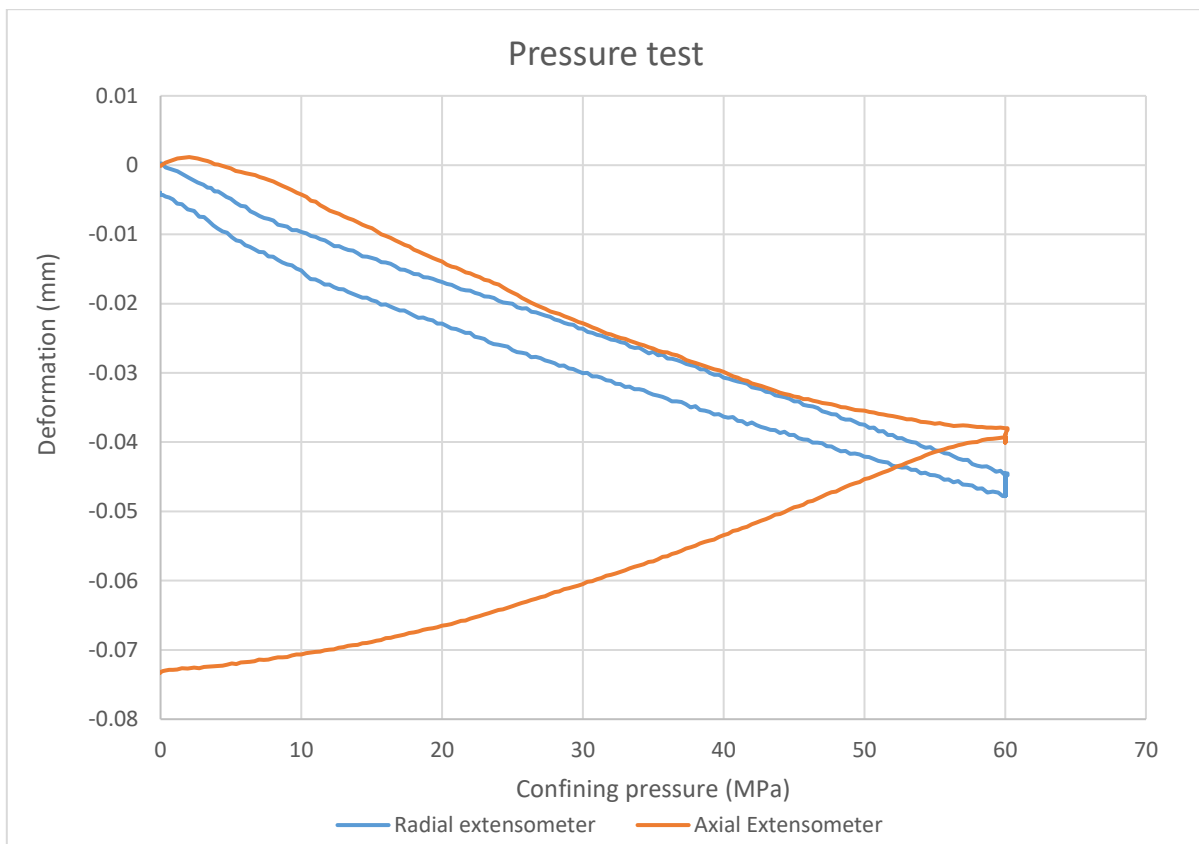


Figure 3.4 Results of pressurized triaxial cell

3.2.4.1 Load test

To investigate if the axial extensometer was malfunctioning, a simple load test was developed. Two metal specimens were used, a steel and an aluminium sample. The advantage of metal is that they are homogeneous and reload cycles do not affect the response. The two specimens had the same dimensions. Strain gauges were attached to the specimen together with the extensometers. This provided data than can be directly compared with the extensometers. Properties of these metals like elastic modulus provide additional data to compare with.

3.2.4.2 Sample preparation.

The specimens had a diameter of 42 mm and a height of 105 mm. Two pairs of strain gauges were attached to each specimen. The horizontal pair measured lateral strain whilst the vertical pair measured the axial strain. Steel platens and a shrink sleeve (local) were added to the specimens. The shrink sleeve allowed the axial extensometer to attach to the sample, without it the extensometer would just slide off. The final setup of the steel sample before testing is shown in Figure 3.5.



Figure 3.5 Specimen with platens and axial extensometer attached.

3.2.4.3 load test procedure

The load test was run in the MTS testing machine with the triaxial cell open. The specimens were loaded to the desired load (aluminium to 150kN and steel to 300 kN) at a loading rate of 1 kN/s. Upon reaching the desired load the machine maintained the same

load for 5 minutes, after which the load was reduced to zero at a rate of 1 kN/s. Two separate systems were used to capture the data. The MTS testing machine does not have the components that allow the connection of strain gauges. This means an independent data capturing system for the strain gauges had to be introduced. The deck system used by the Amsler soft testing machine was used to collect data from the strain gauges. The setup of the two systems is shown in Figure 3.6.



Figure 3.6 Steel sample test setup

The deck system recorded strain readings without a corresponding load. This was because the system was not linked to the MTS load cell. This meant data collection had

to be recorded manually for there to be a comparison between the strain gauges and the extensometer. The data was captured at 20 kN intervals during the loading and unloading phases of the test. The results comparison is shown in Figures 3.7 and 3.8.

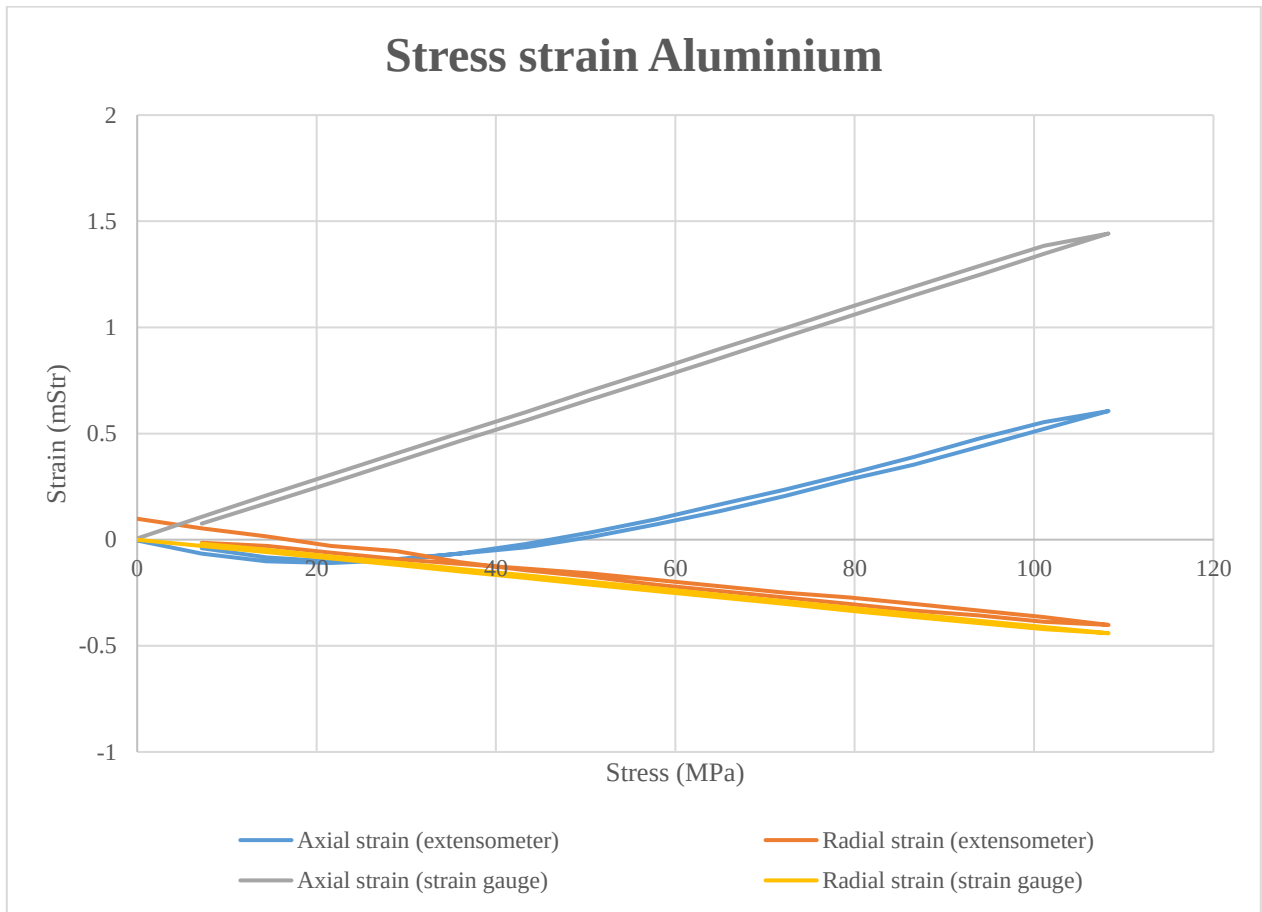


Figure 3.7 Comparison of extensometer to strain gauges (Aluminium sample)

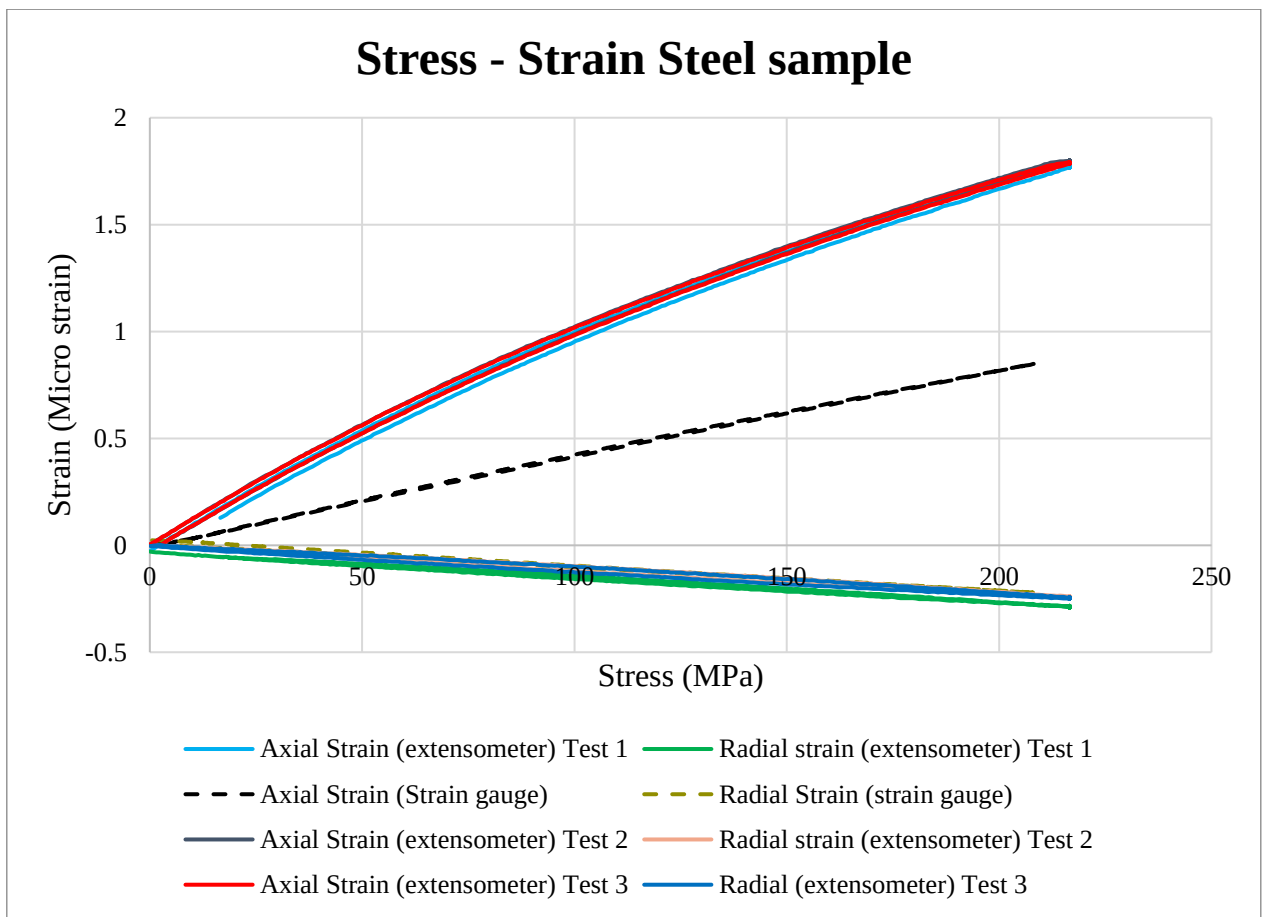


Figure 3.8 Comparison of extensometer to strain gauges (Steel sample)

The difference (in strain readings) between the strain gauge and the axial transducer was more pronounced for the aluminium specimen. The axial strain curve for the extensometer was non-linear. Given that the specimen tested is a metal, obtaining such a curve at low stress is unusual, further suggesting the axial transducer may be malfunctioning. The strain gauges gave an elastic modulus of 73.8 GPa whilst the axial extensometer gave a modulus of 93 GPa. The elastic modulus for aluminium alloy 2024 is 73.1 GPa (Vegas Fastener Manufacturing, 2021). This also suggested that the load cell was operating in the desired manner. Any issues with the load cell would have also affected the stress/strain curves for the strain gauges.

From Figure 3.9, the axial extensometer recorded higher strain reading than the axial strain gauges for all three tests and the readings were not linear. The cause of the non-linear curve is unknown. This may suggest an issue with the axial extensometer. Figure 3.9 shows the axial strain extensometer readings multiplied by a factor of 0.48. Attempts were made to use the factor on other materials and a similar trend was obtained. This factor was used to adjust the axial strain for successful triaxial tests performed with this extensometer. The factor was determined by trial and error. The close agreement between the radial extensometer and the radial strain gauges suggests the radial extensometer was functioning correctly and there was no cross talking between the extensometers.

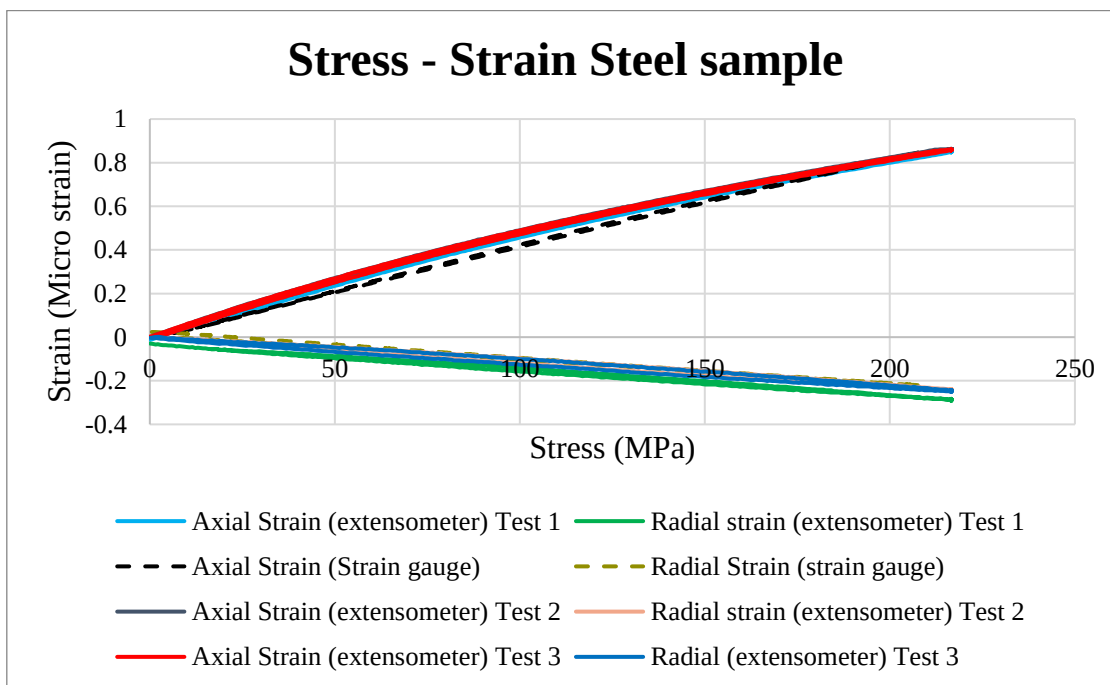


Figure 3.9 Adjusted axial extensometer readings.

3.2.4.4 Influence of shrink sleeve.

Two shrink sleeves were used in this research (American and the local sleeve), it was important to check if they have any influence on the transducer readings and if they were

affecting the performance of the axial transducer. To establish this, simple loading and unloading tests were setup. A chromite sample was loaded up to 97 kN (which is about 50% of the UCS) at 1 kN/s, then held at a constant load for 5 minutes and subsequently dropped back to zero at the same rate. For the first loading cycle the sample had no sleeve attached to it. For the second cycle the local sleeve was attached to the sample. For the final cycle strain gauges were used and the sample was loaded till failure occurred. No confinement was used. Figure 3.10 shows the stress/strain curves for the three cycles.

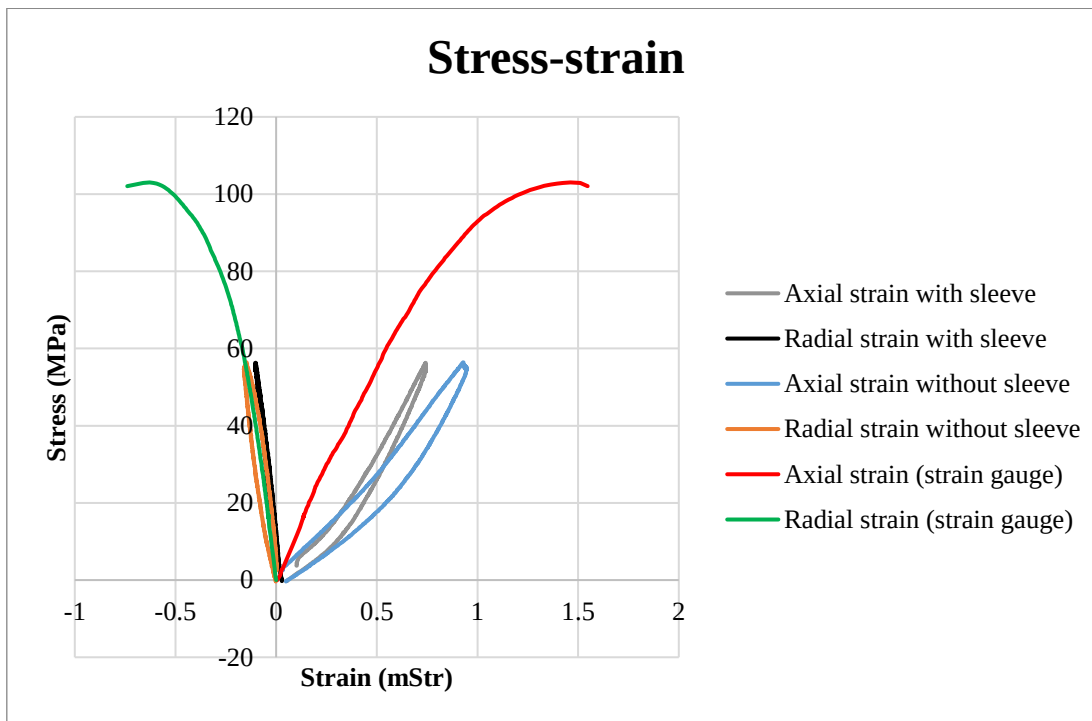


Figure 3.10 Stress/strain curves for no sleeve, with sleeve and with strain gauges

The results of the test suggest the local sleeve tends to reduce the strain readings recorded by the transducers and it is most influential on the axial transducer. The test cycle with the sleeve had a significantly higher modulus (103 GPa) than the one without (71 GPa). This could be caused by slip inside the sleeve. After the second cycle test the

sleeve was removed and strain gauges were attached. It is important to note that cyclic loading a rock specimen induces micro-cracks in the rock and long waiting periods would result in a longer, non-linear region at the beginning of the loading cycle. It was expected that the strain gauged cycle modulus would be close to the modulus of the no sleeve modulus. However, the strain gauged cycle gave a modulus of 101 GPa which is significantly higher than the modulus obtained without the sleeve. This provided more evidence that the axial transducer might have a problem. Figure 3.10 shows the radial strain curves are close to the radial strain gauge curves. The axial strain curves are further away from the axial strain gauge curves, suggesting the axial transducer is recording more strain.

3.2.4.5 The local sleeve

The local sleeve is a thin wall, flexible polyolefin tubing. The rock engineering laboratories at the University of the Witwatersrand have a huge stock of this sleeve, as a result it is most used in triaxial tests. Tests done on the chromite sample suggest that the sleeve tends to reduce the strain recorded by the transducers. Additional tests were done using the sleeve on an aluminium specimen.

The purpose of these tests was to investigate the influence of the local sleeve on the transducers under confinement. The aluminium specimen with the local sleeve was loaded to 200 kN at rate of 1 kN/min and unloaded at the same rate. In between the loading and unloading cycles the load was held constant for 5 minutes. The tests were conducted at confinements of 0 MPa, 10 MPa and 60 MPa. The specimen was loaded to 150 kN for the specimen without confinement. The results of the tests are shown in Figure

3.11. The strain gauges were from a previous test done on the specimen (from Figure 3.7).

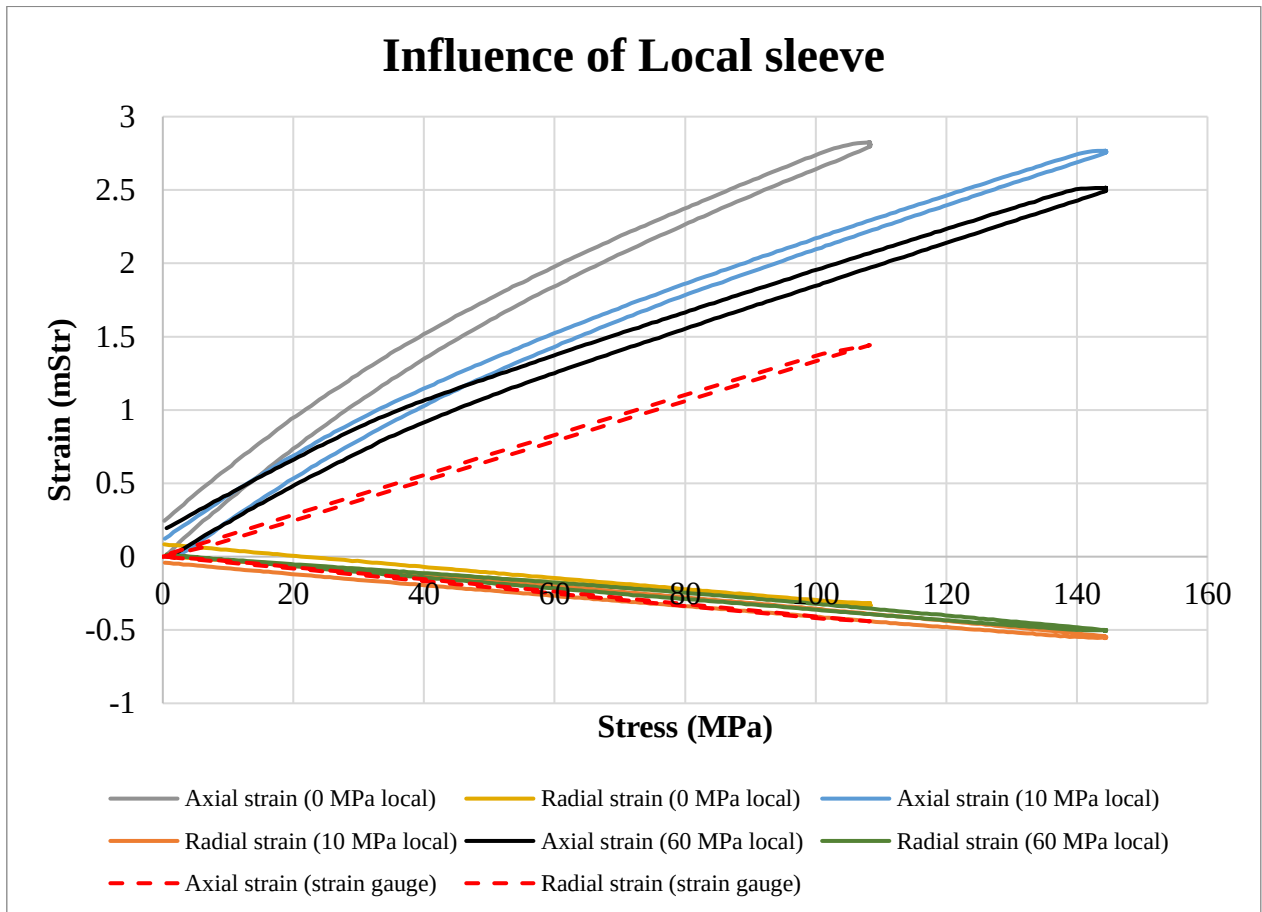


Figure 3.11 Influence of the local sleeve under different confinements

The results from the tests suggest that the strain per unit load recorded by the axial transducer tends to decrease with increasing confinement. Unfortunately strain gauge comparison was only possible at zero confinement. The use of an external data collection unit is not possible with the triaxial cell closed.

3.2.4.6 The American sleeve

There is no information about the material composition of this sleeve. The sleeve is not flexible and has to be stored in vertical tubes that prevent it from bending. Experience from previous triaxial tests done shows the American sleeve handles higher confinements (> 30 MPa) better than the local sleeve. However, there is limited stock and the sleeve is usually reserved for tests that involve high confinement. Figure 3.12 shows the two sleeves.



Figure 3.12 Local (left) and American sleeve (right)

The same tests done on the local sleeve were repeated on the American sleeve and the results are shown in Figure 3.13.

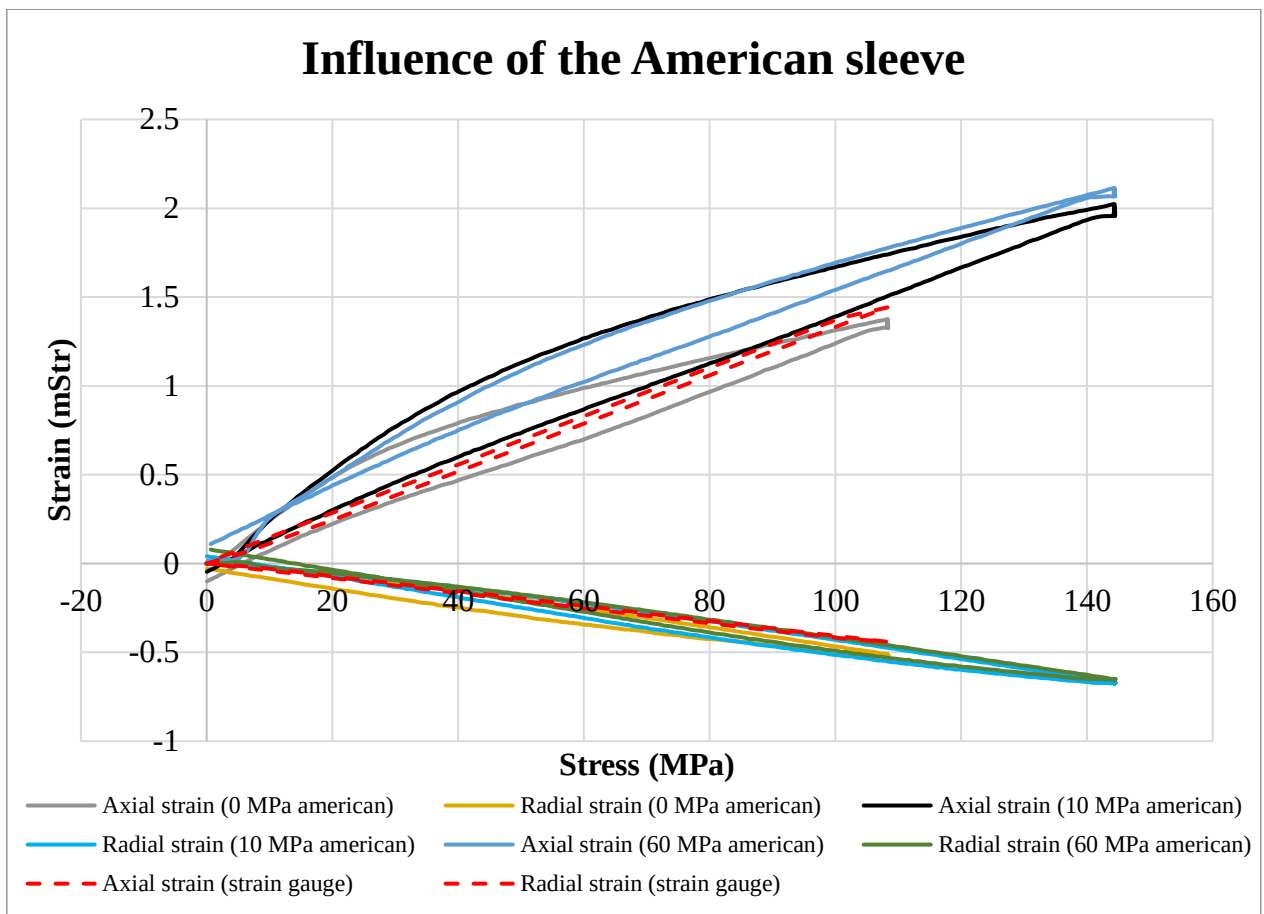


Figure 3.13 Effect of confinement on the American sleeve

The American sleeve's axial strain per unit load increases with increasing confinement. The loading cycle (axial strain) for each confinement is not linear, however, the unloading cycles seem to be linear and parallel to the strain gauges. The sleeve seems to have little effect on the radial extensometer. A comparison of the two sleeves is shown in Figure 3.14.

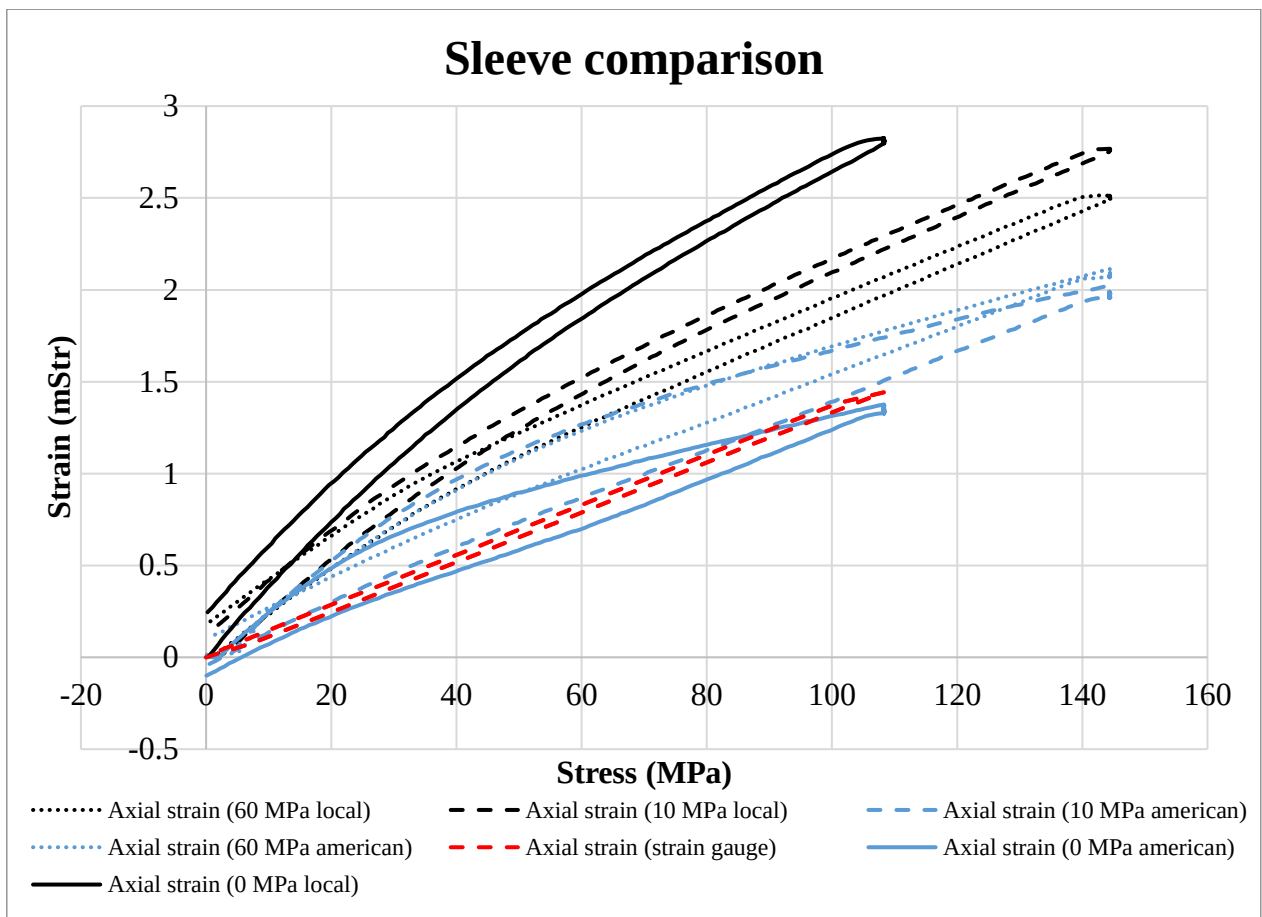


Figure 3.14 Effect of confinement on the two sleeves

Two colours were used to represent the two sleeves. Black represents the local sleeve and blue represents the American sleeve. Different patterns were used to represent different confinements, for example a solid curve represents no confinement. The radial strain curves were removed as the emphasis is on the axial extensometer and to reduce clutter on the chart. Also, the sleeves have shown little influence on the radial extensometer. The American sleeve seems to have a low strain per unit load compared to the local sleeve. Both sleeves produced non-linear curves in the loading cycle with American sleeve curves having the most curvature. Surprisingly, the unloading cycle curves of the American were linear and parallel to the strain gauges, unlike the local

sleeve's unloading path which is similar to its loading path. The cause of this linear behaviour in the unloading phase is unknown, however, the interest of this research lies in the loading cycle. All tests conducted for the purpose of this research involve loading, which is why it was concluded that both sleeves produced non-linear curves. The evidence from the tests conducted suggests that the sleeve does influence the axial extensometer reading. It is important to note that the sleeves are causing more deformation to be recorded than what is occurring. This may indicate that the sample is not slipping inside the sleeve. The influence is difficult to compute as it varies with each specimen (in the case of rocks). However, the evidence still suggests that there may be an issue with the axial extensometer.

3.2.5 Axial extensometer calibration

After all the testing done on the extensometers, there was enough evidence to motivate the inspection of the MTS machine by technicians. The third-party representatives of MTS in South Africa (IMP) performed a calibration check of the axial extensometer. The following section describes the observed calibration process of the axial extensometer.

3.2.5.1 Calibration procedure

The MTS multi testing software was switched to calibration mode. The axial extensometer was attached to the micrometer gauge. An external cable was used to link the extensometer with the MTS machine. The handle at the top of the digital micrometer was used to adjust the distance between the platens. Below the handle (Figure 3.15) is a digital micrometer screen that shows how much distance the platens have been adjusted by.

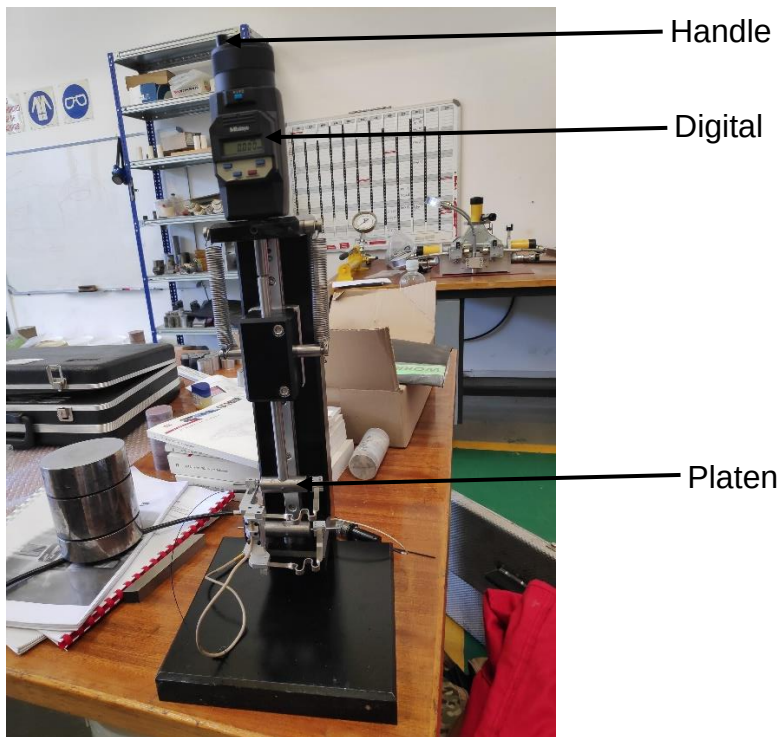


Figure 3.15 Micrometre screw gauge

The platens were adjusted at 200 μm intervals until reaching the upper limit of the extensometer. At each interval corresponding readings from the MTS machine were recorded. The process is repeated from the upper limit to the lower limit of the extensometer. Figure 3.16 shows a plot of the micrometre reading against the average MTS readings. An average of three readings were taken per micrometre interval.

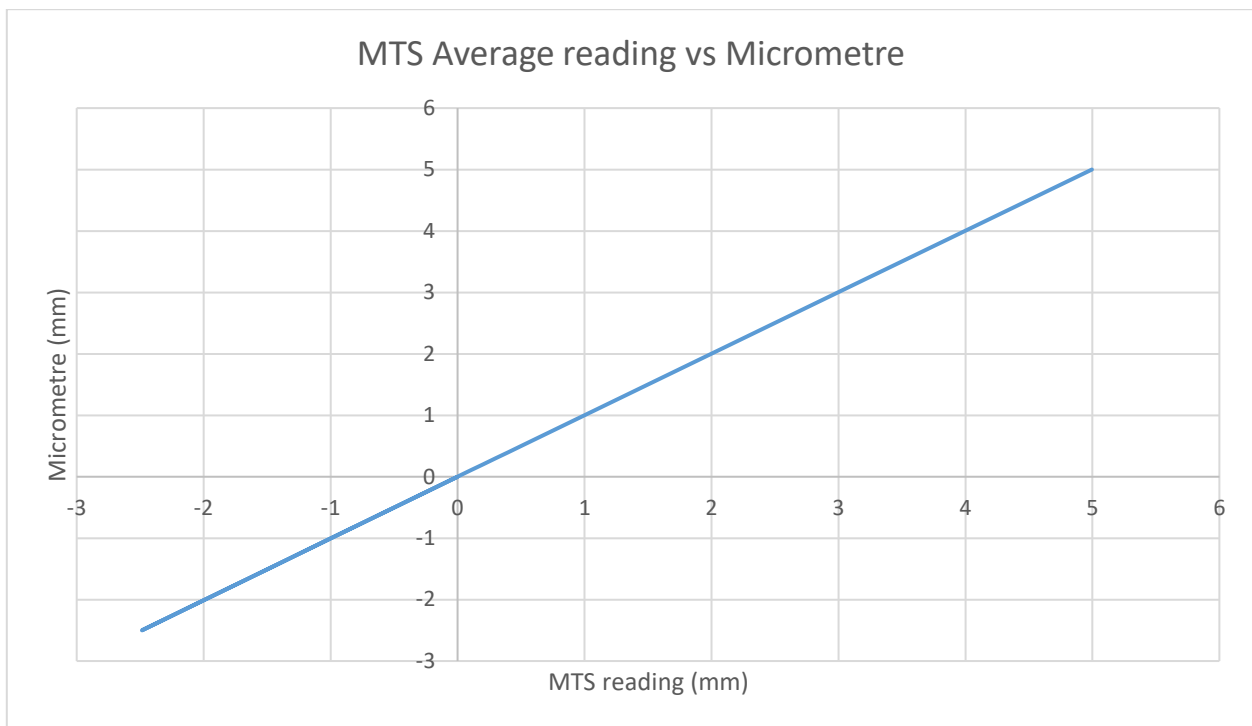


Figure 3.16 Micrometre readings vs MTS reading

The plot shows the extensometer was producing linear results across its extension range. According to the technician, the extensometer was behaving as per company standards and re-calibration was not required. This also suggested that the axial extensometer was fully functional. However, the calibration check does not include confinement and the technician was not aware of method to check the extensometer behaviour under confinement. The recommendation given by the technician was to contact MTS. Efforts to contact MTS proved futile. It was suggested to try a different extensometer. IMP provided a new extensometer which was used to perform a few triaxial tests and no issues were observed.

3.2.6 New axial extensometer

Triaxial test results showed that the new extensometer was behaving properly. A pressurized triaxial test was done on the new extensometer and the results were compared to the existing extensometer as shown in Figure 3.17. The new extensometer has a smaller error (within the error limit ± 0.03 mm) at the end of the test.

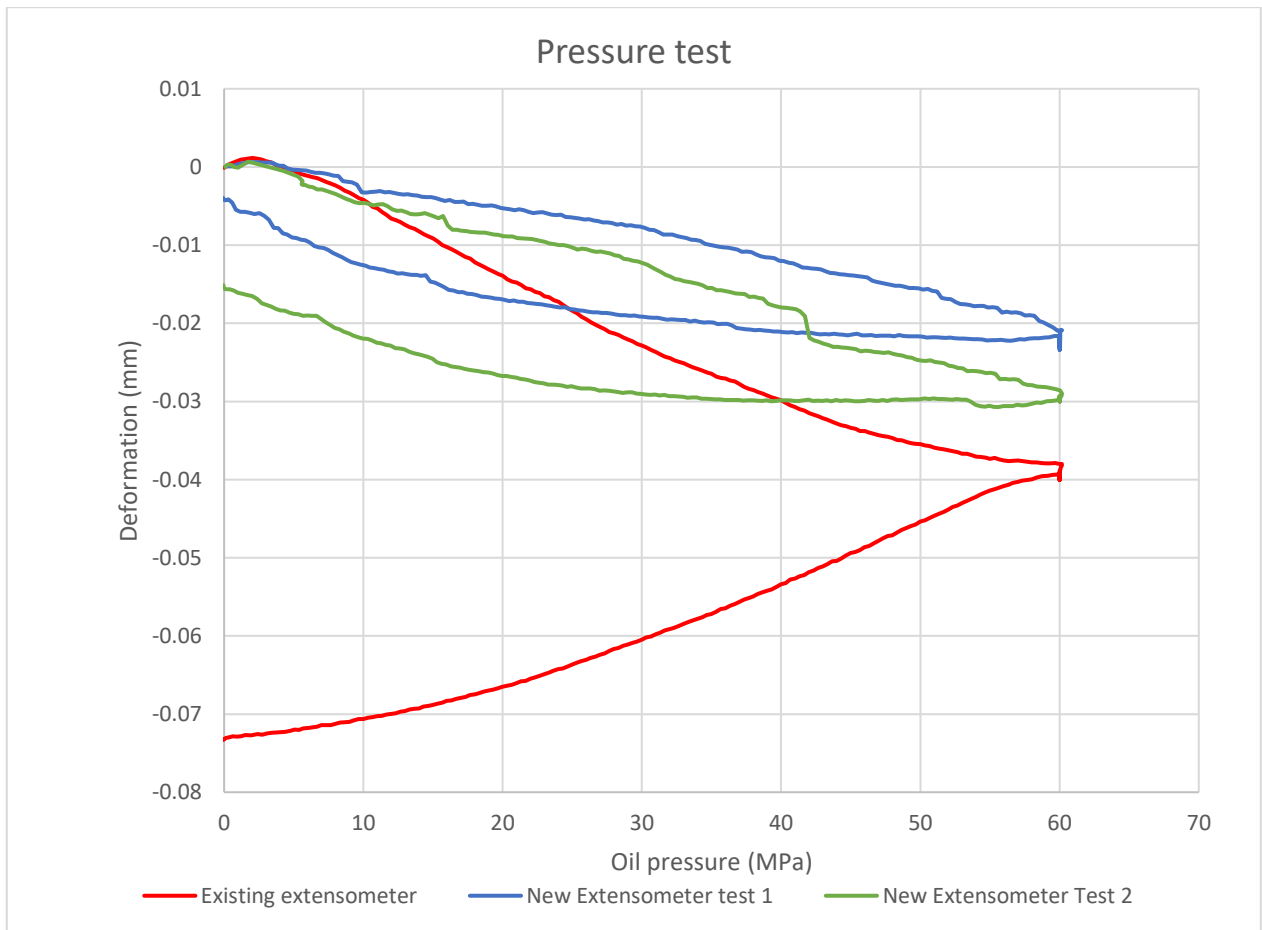


Figure 3.17 Existing extensometer vs new extensometer.

The results of the pressure test showed improved accuracy with errors within acceptable margins. Additional tests on steel and aluminium samples were also performed and the

results are shown in Figures 3.18 and 3.19, respectively. The test procedure described in section 3.2.4.3 (load test) was used.

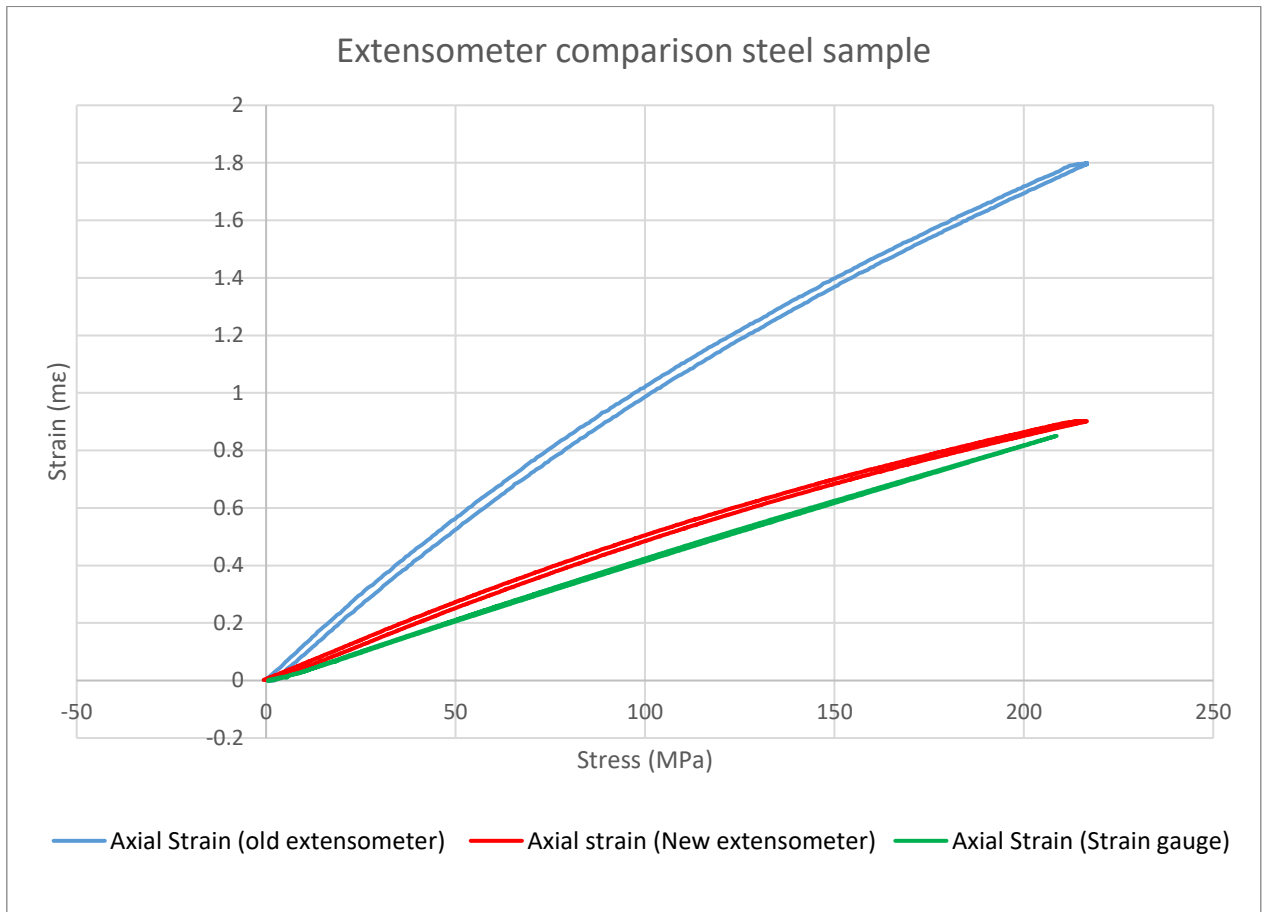


Figure 3.18 Comparison of new and old extensometer

A significant improvement was observed with the new extensometer. The stress/strain curves of axial strain gauges and the axial transducer are very close with little difference between them. The small difference between the two curves was expected as strain gauges are more accurate than the extensometers. The curved nature of the new extensometer's stress/strain curve was concerning. However, this curved nature is uniquely produced for steel samples. Figure 3.19 shows the comparison of the two extensometers on an aluminium sample.

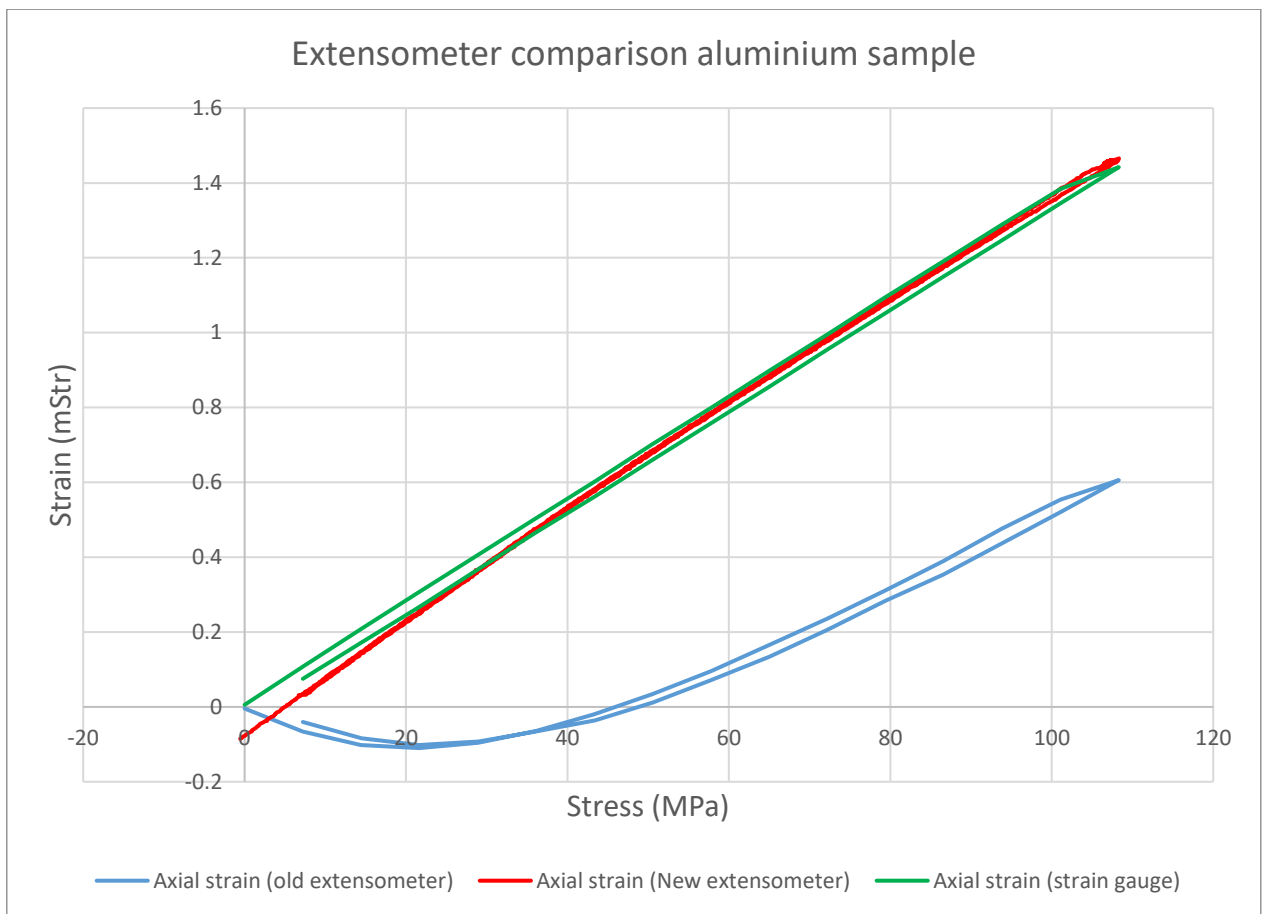


Figure 3.19 Comparison of test results from the new and old extensometer on an aluminium sample

The results from the new extensometer almost perfectly matched the results from strain gauges. The stress/strain for the new extensometer was linear in both the loading and unloading phases of the test. This provided more conclusive evidence suggesting that the old extensometer had an issue. The two load tests performed on the steel and aluminium involved the use of a sleeve (local sleeve). The resulting curves do not show any significant influence from sleeve. Therefore, in this research the influence of the sleeves is considered insignificant. No load tests were performed on the American sleeve because of the limited stock. The new extensometer was used to perform a series of post-peak triaxial tests on different rock types, with both sleeves at different confinements, the results

were satisfactory and comparable to those found in literature. After all the testing done on both extensometers, it was decided to proceed with the rock testing with the new extensometer and the old extensometer was declared faulty. However, the cause of the axial extensometer issues is still unknown and there are no guarantees that the issue will not arise in the future.

3.3 Sample preparation and test procedure

Most of the tests performed in this research were triaxial tests performed on the MTS machine. Other tests performed include the UCS test and the indirect tensile test. The specimen preparation process and test procedures are described in the following section.

3.3.1 Rock samples

The rock samples used in this researched were provided by Impala Mine located in the western limb of the Bushveld Complex. Four rock types were provided, and Table 3.1 summarizes their details.

Table 3.1 Summary of rock samples

Rock type	Borehole depth range (m)	Diameter (mm)	Number of samples
Anorthosite	798.46 – 1836.82	47.3	25
Chromitite	813.91 – 1854.67	46.9	25
Pegmatoid	1194.12 – 1818.27	47.1	25
Pyroxenite	798.46 – 1836.82	47.0	24

3.3.2 Specimen preparation

All the test specimens were prepared in line with the ISRM guidelines (Ulusay and Hudson, 2007). Test specimens used for Uniaxial Compressive Strength (UCS) and triaxial tests were right circular cylinders and had a height to diameter ratio between 2.0 and 3.0. The ends of the specimens were flat to $\pm 0.01\text{mm}$ and the sides were smooth and straight. Some of the core samples had to be cored down to a diameter of 42 mm. This was due to the limitation of the radial transducer. Some rock specimens (Chromitite and pegmatoid) experienced radial deformations that exceeded the maximum deformation reading range possible on the radial transducer. Specimens used for UCS had two pairs of strain gauges attached on the curved side as shown in Figure 3.20. One pair measured the axial strain whilst the other pair measured the radial strain.



Figure 3.20 Strain gauged samples

3.3.3 Indirect tensile test (Brazilian test)

The test specimens were prepared according to ISRM standards, with the right cylinders having a height of half the diameter. The specimens had a diameter of 47 mm. A total of 5 tests were done per rock type. Impala Mine provided results from indirect tensile tests conducted at the mine site. Results from the laboratory were compared to the results provided by the mine. The apparatus of the indirect tensile test are steel loading jaws and a machine able to apply and measure the compressive load to the apparatus. Figure 3.21 shows a test specimen inside the indirect tensile test cell.



Figure 3.21 Indirect tensile test setup showing the steel jaw cell

3.3.3.1 Indirect tensile test procedure

The test specimen was placed between the steel jaws as shown in Figure 3.21. The steel jaw cell was then placed between the platens of the loading machine. The MTS criterion testing machine was used to provide the load at a constant rate of 200 N/s until failure. The samples failed by splitting, initiating at the centre of the specimen and extending across the diameter in the direction that the load was applied as shown in Figure 3.22.

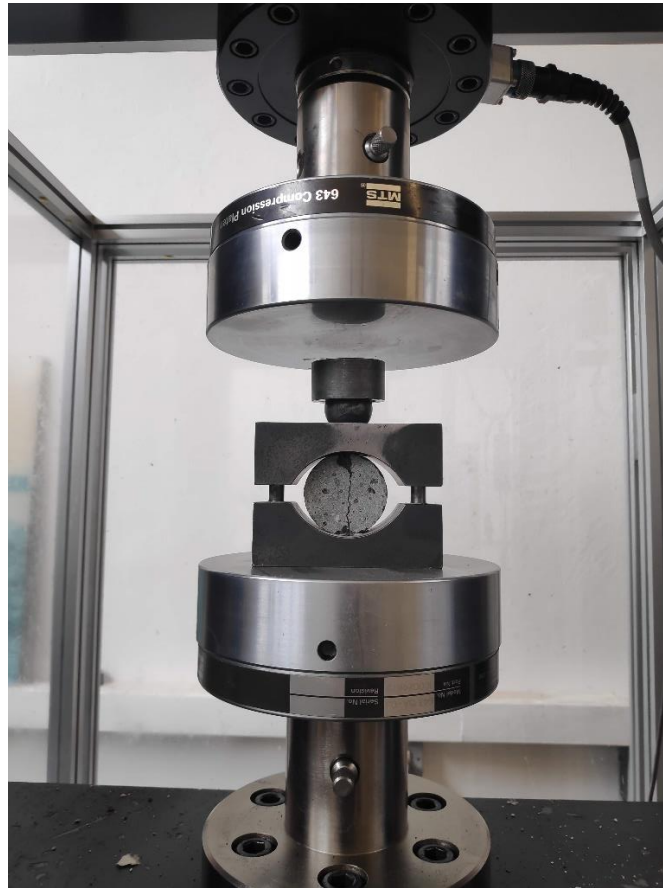


Figure 3.22 Specimen in the MTS criterion testing machine

The load at which the specimen fails was used in Equation 3.3 to determine the tensile strength of the rock.

$$\sigma_t = 0.636 \frac{P}{Dt} \quad (3-3)$$

Where σ_t is the tensile strength,

P is load at failure (N),

D is the diameter of the specimen (mm) and

t is the thickness of the specimen (mm).

3.3.4 Tilt test

Tilt tests were conducted to estimate the friction angle of the plane where two lithologies met. These were required to calibrate the interfaces in the numerical models presented in Chapter 4. Three interfaces exist in the model and these are:

- Anorthosite-pegmatoid
- Pegmatoid-chromitite
- Chromitite-pyroxenite

Samples that were prepared for triaxial and UCS tests were used for this test. The test does not affect the integrity of the samples. This means the samples could be re-used for other tests. The apparatus required for this test involves a tilt table and protractor. Three samples from two different rock types are required to perform the test. The samples are stacked on the tilt table as shown in Figure 3.23. The two samples at the base are same rock type and the sample at the top is a different rock type. Turning the handles clockwise would result in the one end of the tilt table rising. The handles were slowly rotated till the sample on top started sliding. The angle between the horizontal and the raised platform would be taken as the friction angle. Figure 3.23 shows the tilt test setup. Table 3.2 shows the results of the tests.

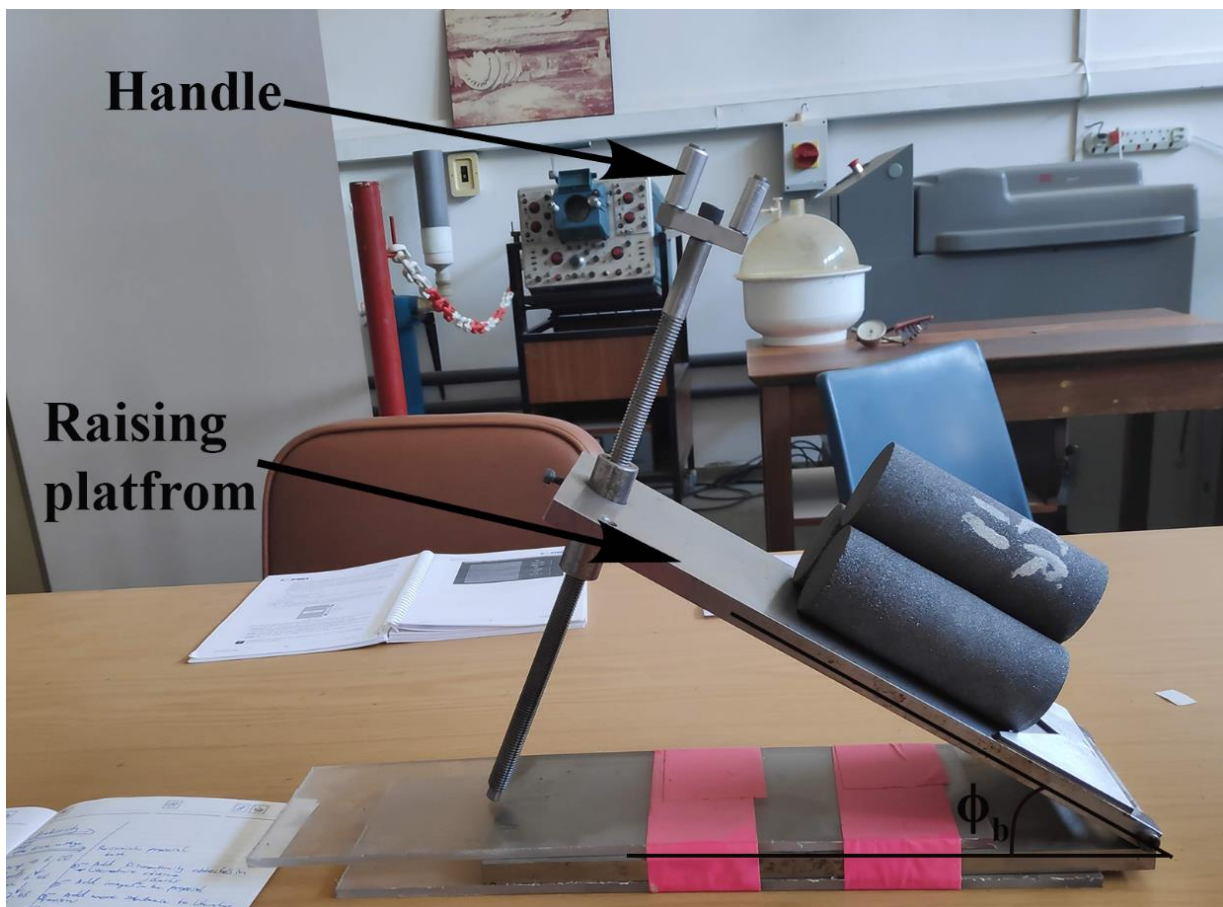


Figure 3.23 Tilt test setup

Table 3.2 Tilt test results

Interface	Friction angle
Anorthosite-pegmatoid	30°
Pegmatoid-chromitite	33°
Chromitite-pyroxenite	35°

3.3.5 Uniaxial Compressive strength test

The uniaxial compressive strength tests were done using the Amsler testing machine shown in Figure 3.24. The Amsler is a soft loading machine using hydraulics and has a loading capacity of 2000 kN. The machine is manually operated using the analogue input dials to guide the loading rate. The machine is load controlled and violent failure of specimen usually takes place once the peak strength is exceeded. This is caused by the soft nature of the testing machine and loading mechanism. The hydraulic piston sits at the base of the loading frame.

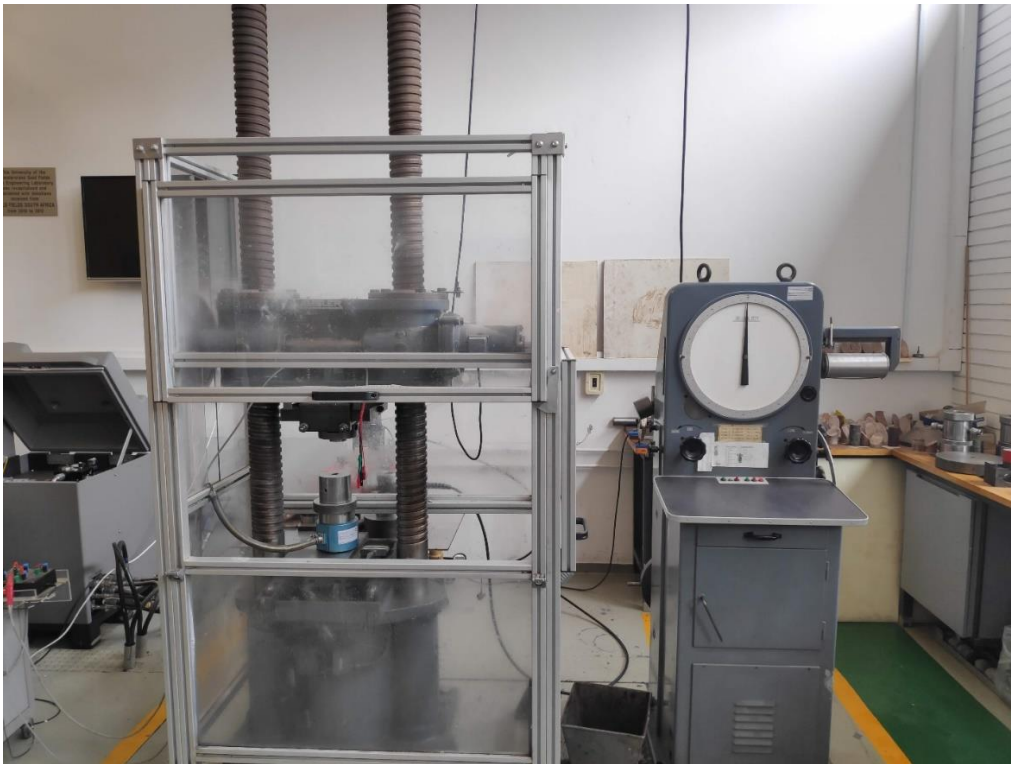


Figure 3.24 Amsler compression testing machine

3.3.5.1 UCS test procedure

The strain gauged specimen is connected to a signal amplifier (Figure 3.25) which is linked to the computer, where the signals (from the instrumentation) are converted and stored in a digital format. The sample is placed in the centre of the bottom platen. The top platen is lowered till it meets the sample. A small load is then applied on the sample to check if the strain gauges are sending a signal to the data acquisition unit (Figure 3.25). A pre-load of about 20 kN is applied. The specimen is then loaded at a rate of 0.5 kN/s until failure. Duration of the tests were in the range recommended by the ISRM guidelines (Ulusay and Hudson, 2007). Three samples per rock type were tested. Impala Mine also provided UCS tests done elsewhere. Figure 3.25 shows the data acquisition unit and the signal amplifier.

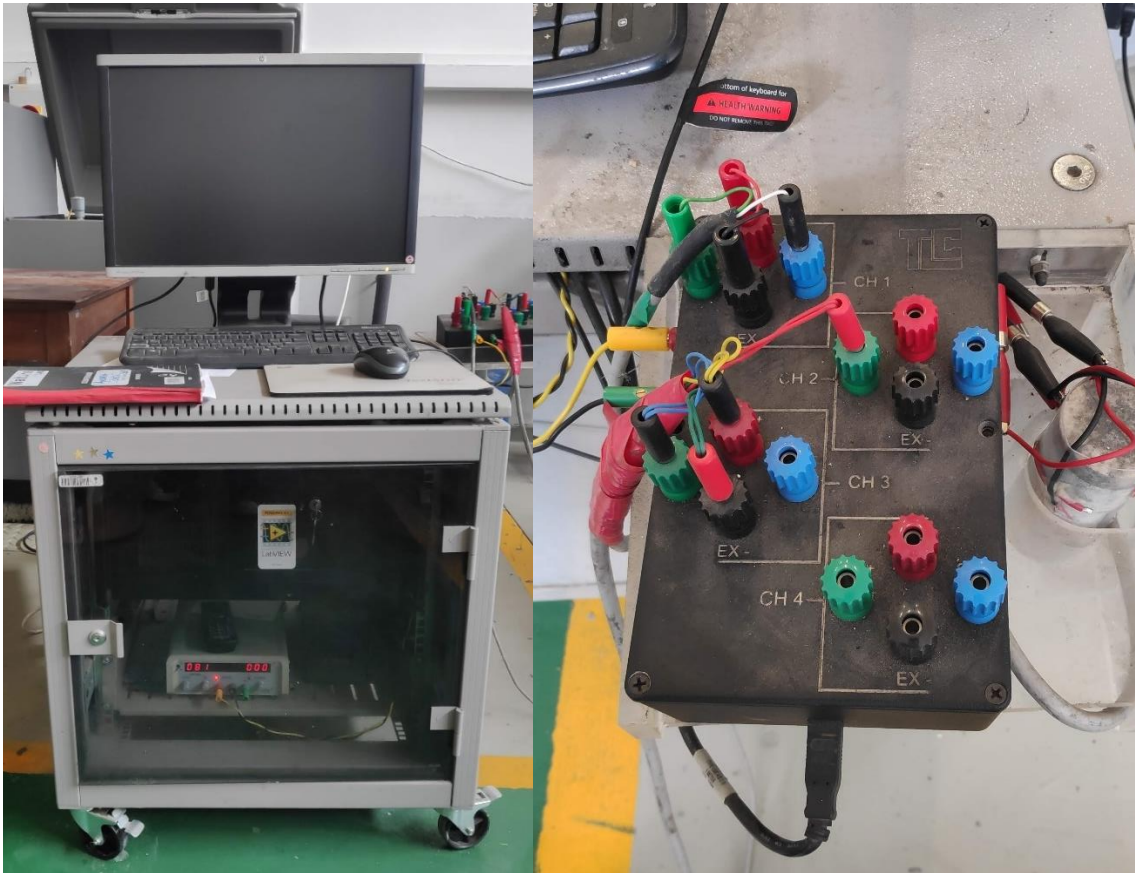


Figure 3.25 Data acquisition unit on the left and on the right the strain gauge amplifier.

3.3.6 Triaxial testing using the MTS servo-controlled testing machine.

Triaxial testing was conducted using the MTS 815 servo-controlled testing machine (Figure 3.26). This is a stiff testing machine. The combination of the machine's stiffness and servo-control gives the MTS 815 the capability to investigate the post peak properties of most rock-types tested at a diameter of about 50 mm. The information obtained from these tests will be used to calibrate numerical model input parameters.

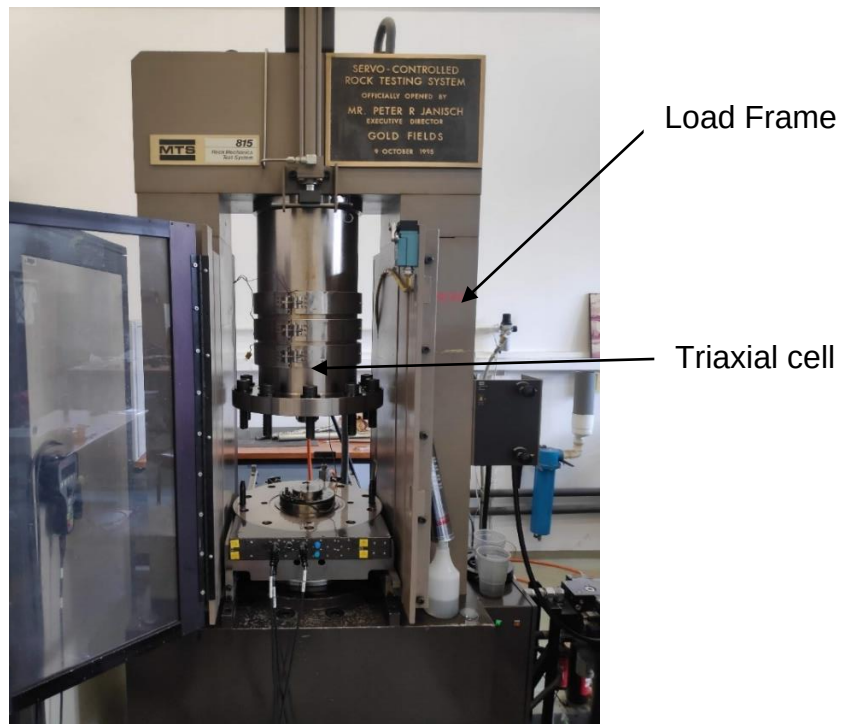


Figure 3.26 The MTS 815 servo-controlled testing machine

3.3.6.1 Description of apparatus

The MTS 815 testing machine comprises of the following components:

Loading frame: The highly stiff loading frame assists in the fracture control capability during post-peak studies. The load frame assembly comprises of a crosshead mounted on two rectangular columns that are bolted to the base producing a semi-rigid free-standing frame. Figure 3.26 shows the load frame of the MTS 815 testing machine.

Hydraulic power unit (HPU): The hydraulic power unit provides the hydraulic pressure to the piston. It initially provides 1 MPa at start up, and the pressure can be increased to a maximum of 21 MPa. The HPU requires an external cooling unit to maintain the oil temperature at optimal operating temperature. Figure 3.27 shows Hydraulic power unit.



Figure 3.27 Hydraulic power unit

Servo-valve: The servo-valve is mounted on the actuator manifold. Its primary functions are to regulate oil pressure, flow rate and direction of flow. The servo-valve allows the MTS 815 machine to quickly adjust the position piston. This is crucial during post-peak tests as the machine needs to respond rapidly to any rock instability that develops during the test. The servo-valves are shown in Figure 3.28.

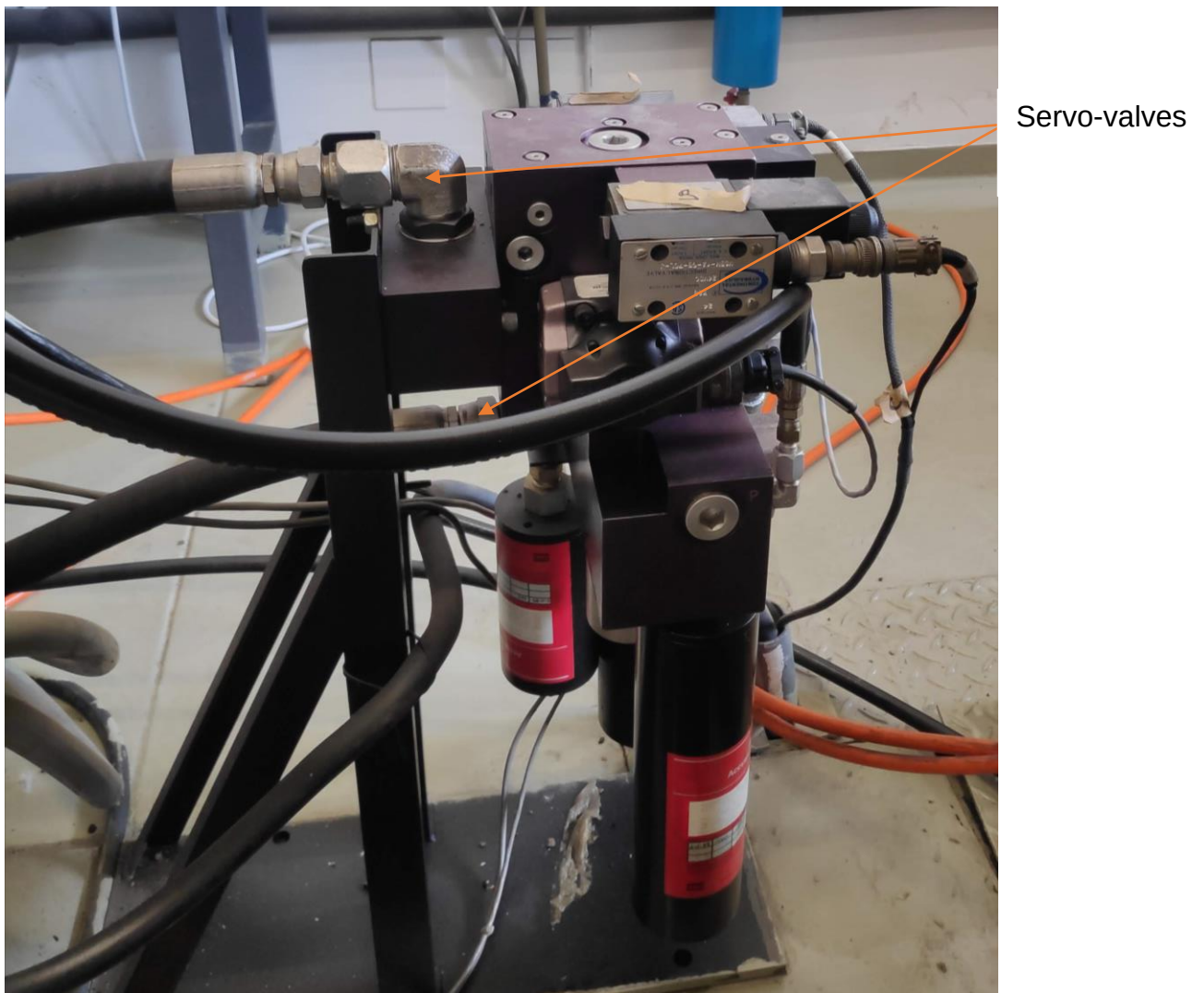


Figure 3.28 Servo-valves

Triaxial cell: The triaxial cell assembly houses pressurized oil when performing triaxial tests. The pressurized oil provides the confining stress. The triaxial cell has a 140 MPa capacity. Cap screws are used to hold the triaxial cell to the base plate (Oniyide, 2015). The cell triaxial conditions simulate a stress environment similar to what rocks experience underground. Figure 3.26 shows the triaxial cell.

Confining pressure intensifier: The confining pressure intensifier supplies oil and pressurizes the triaxial cell. The unit also acts as a reservoir. An air pump built in the pressure intensifier is used to pump oil into the triaxial cell. Compressed air is used to empty the cell returning the oil to the confining pressure intensifier's reservoir. The confining pressure intensifier allows for the servocontrol of the confining oil. Figure 3.29 shows the confining pressure intensifier.



Figure 3.29 Confining pressure intensifier

Control System: The MTS flex 60 provides high-speed closed-loop control, function generation, transducer conditioning and data acquisition to address the full spectrum of testing needs (MTS, 2004). The MultiPurpose TestWare (MPT) software allows the operator to custom design and automate test procedure. Figure 3.30 shows the MTS flex 60.



Figure 3.30 MTS Flex 60

Extensometers (transducers): The extensometer measures the deformation in the specimen during testing. The MTS 815 uses two extensometers, the axial and the radial extensometer. The axial extensometer has a gauge length of 50 mm and an extension range of 7.5 mm. The radial extensometer has an extension range of 12 mm. The axial and radial extensometers have a calculated uncertainty of ± 0.03 mm and ± 0.008 mm respectively. Figure 3.31 shows the two extensometers.

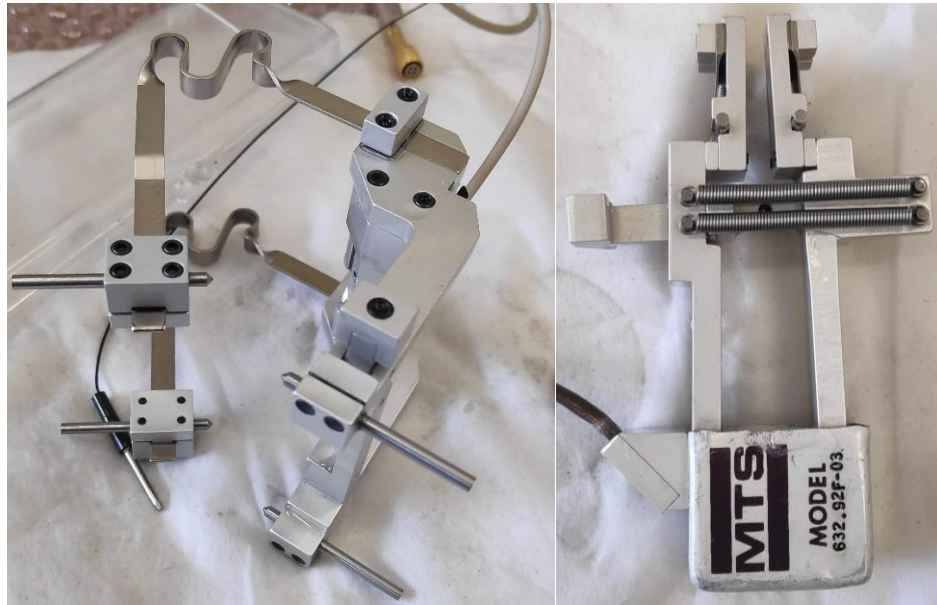


Figure 3.31 Axial extensometer on the left and the radial extensometer on the right

3.3.6.2 Post-peak triaxial test procedure

The extensometers are attached to the sample as shown in Figure 3.32. The triaxial assembly is then lowered and screwed to the base plate. The air pump is used to fill up the triaxial cell with oil. The confinement is increased at 1 MPa/s until the desired confinement is reached.

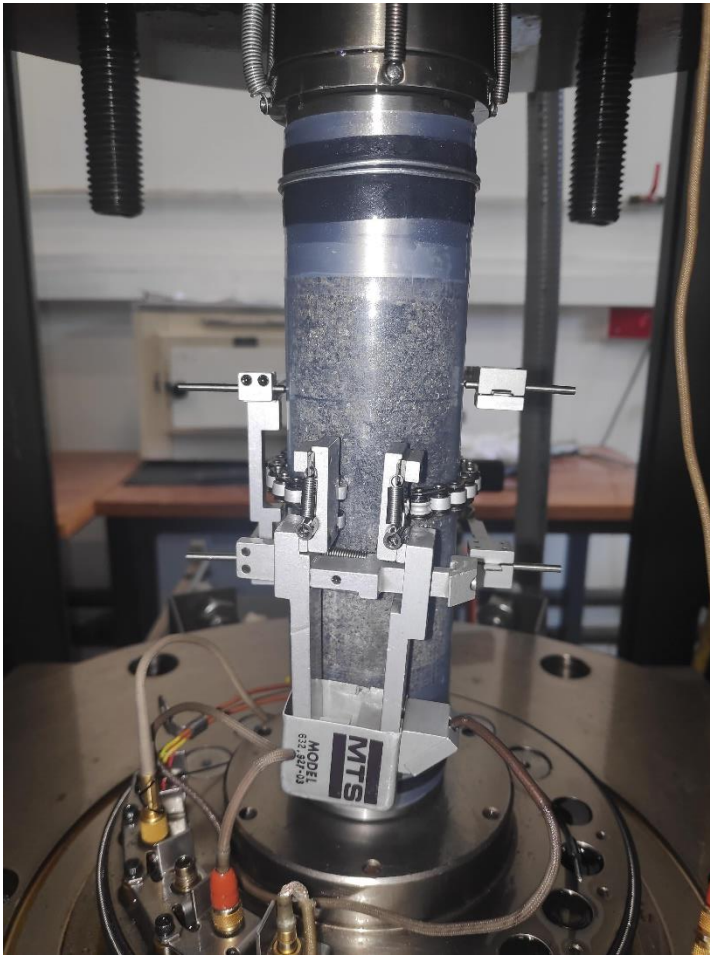


Figure 3.32 Specimen ready for testing (American sleeve)

The MPT procedure used to execute the triaxial tests is shown in Figure 3.33. The change in extensometer output from strain to engineering units necessitated altering the test procedure recommended by ISRM. The triaxial tests were conducted at confinements of 10 MPa, 20 MPa and 40 MPa except for anorthosite that was conducted at confinements 10 MPa, 20 MPa and 30 MPa. The reason for the lower maximum confinement in the anorthosite samples was because violent failure took place at a confinement of 40 MPa. A pre-load of 15 kN was applied to seat the specimen to the loading platens. The MPT procedure used to perform triaxial tests is shown in Figure 3.33.

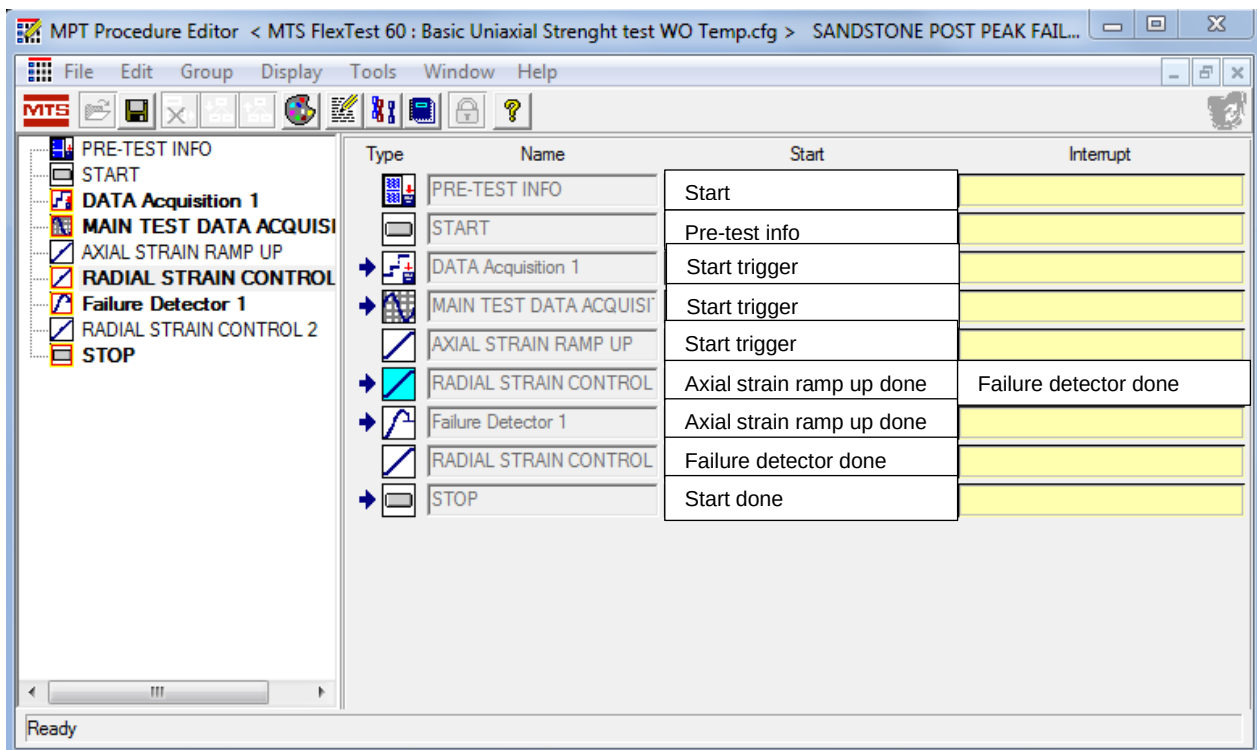


Figure 3.33 MPT procedure used to perform triaxial tests on the MTS machine

The control mode was set to axial strain control and the specimen was loaded at an axial deformation rate of 0.01 mm/min until the changeover point at 0.1 mm to radial strain control. The radial strain was set to deform the specimen at a rate 0.14 mm/min until it reached the peak strength. At peak strength a failure detector was activated triggering the slowing of the radial strain to 0.09 mm/min. It should be noted that these values worked in most cases. Some rocks required unique values to be used.

3.4 Test results

This section presents the results obtained from the laboratory tests and data provided by the mine. The laboratory tests were conducted to get insights in the post-peak behaviour of rocks found in the UG2 reef. The results obtained served as input parameters for the numerical model.

3.4.1 Post-peak investigations

The results of triaxial tests conducted in the laboratory are presented in this section. The post-peak investigations were done at confinements of 10 MPa, 20 MPa and 40 MPa for all rock types except for anorthosite that was done at confinements of 10 MPa, 20 MPa and 30 MPa. A minimum of three tests were conducted at each confinement. In the stress-strain curves, the confinement for each stress-strain curve is represented by a separate colour.

- Blue represents 10 MPa,
- Orange represents 20 MPa and
- Black represents 40 MPa (for anorthosite black represents a confinement of 30 MPa).

3.4.1.1 Pyroxenite results

Figure 3.34 shows the stress-strain curves for pyroxenite at three different confinements. At 10 MPa, pyroxenite exhibited class II behaviour, however, results from test sets 2 and 3 (shown in appendix A) indicate a class I/class II type of behaviour at 10 MPa. At 20 MPa the behaviour transitions to class I and maintains the same behaviour at 40 MPa. All samples tested were obtained at depths greater than 1000 m. At higher confinements open cracks close and the rock exhibits class II behaviour. However, the opposite is observed. Pyroxenite is known to be a class I rock type (Oniyide, 2015). It is possible that the confinements used in this research were not high enough to cause a change in behaviour. However, it is not clear why transitional behaviour was observed at 10 MPa. Observations made by Tarasov and Potvin (2013) suggest that increasing confinement can lead to contradictory (unconventional) rock behaviour within different ranges of confinement. A rock displaying Class II behaviour at low confinement could change to

Class I at high confinement and with further increase in confinement could change back to Class II (Tarasov and Potvin, 2013).

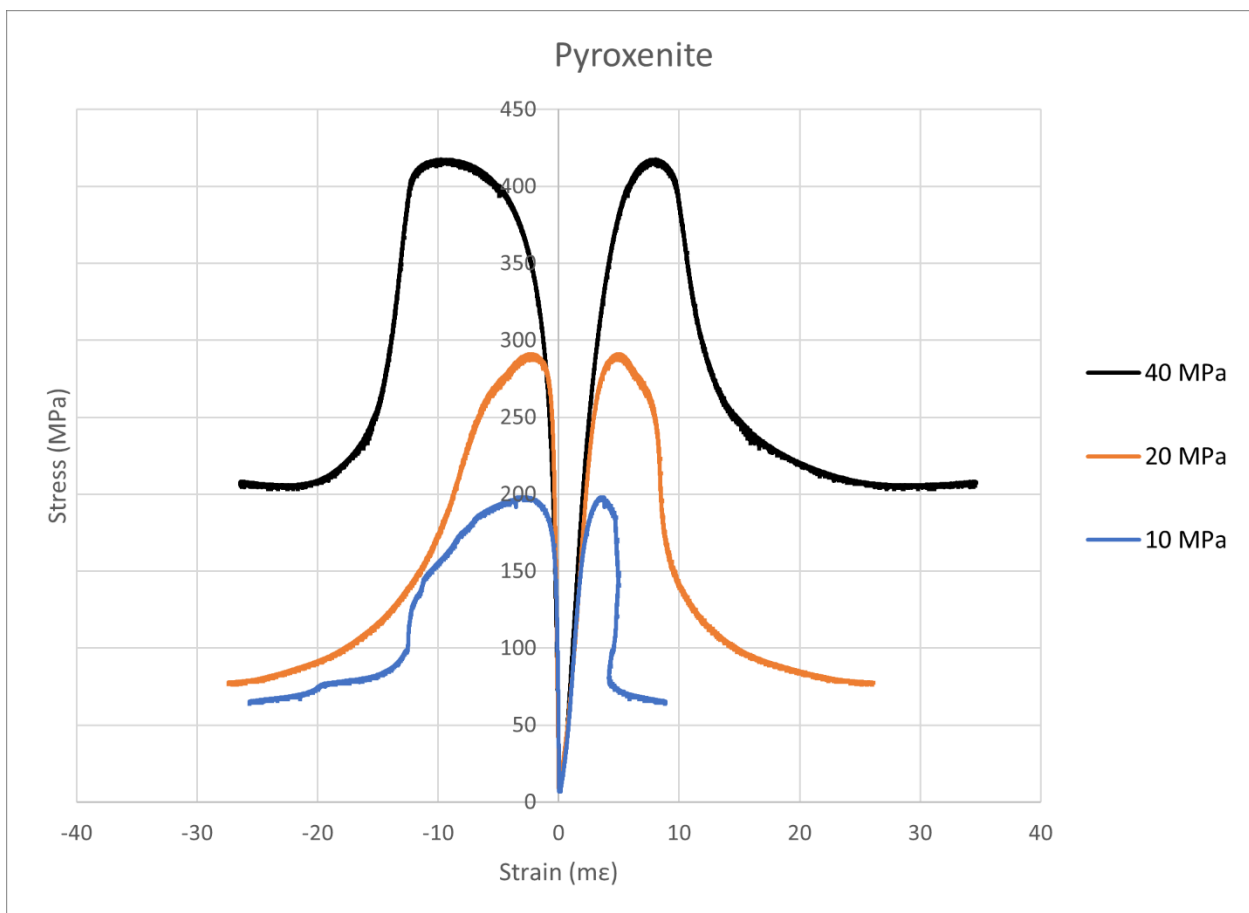


Figure 3.34 Stress-strain curves for pyroxenite

3.4.1.2 Pegmatoid results

Pegmatoid exhibited class I behaviour at confinements of 10 MPa and 20 MPa. At 40 MPa it showed class I/class II type (Figure 3.35) of behaviour, suggesting it was transitioning to class II. Results from pegmatoid set 2 (shown in appendix A-3) also support this observation. However, no tests were conducted at confinements higher than 40 MPa, hence no conclusive evidence showing that pegmatoid does eventually exhibit class II at higher confinements. Work done by Tarasov and Potvin (2013) has illustrated situations

where the continual increase of confinement results in a change of rock behaviour. In addition, results from pegmatoid set 3 (appendix A-4) shows class I behaviour at 40 MPa. It is important to note that the specimens tested were obtained at depths in the range 957.84 m – 1000.79 m. This is at the changeover depth observed by Watson (2010) and it's not clear what rock behaviour is expected. Figure 3.35 shows the stress-strain curves at confinements of 10 MPa, 20 MPa and 40 MPa for pegmatoid.

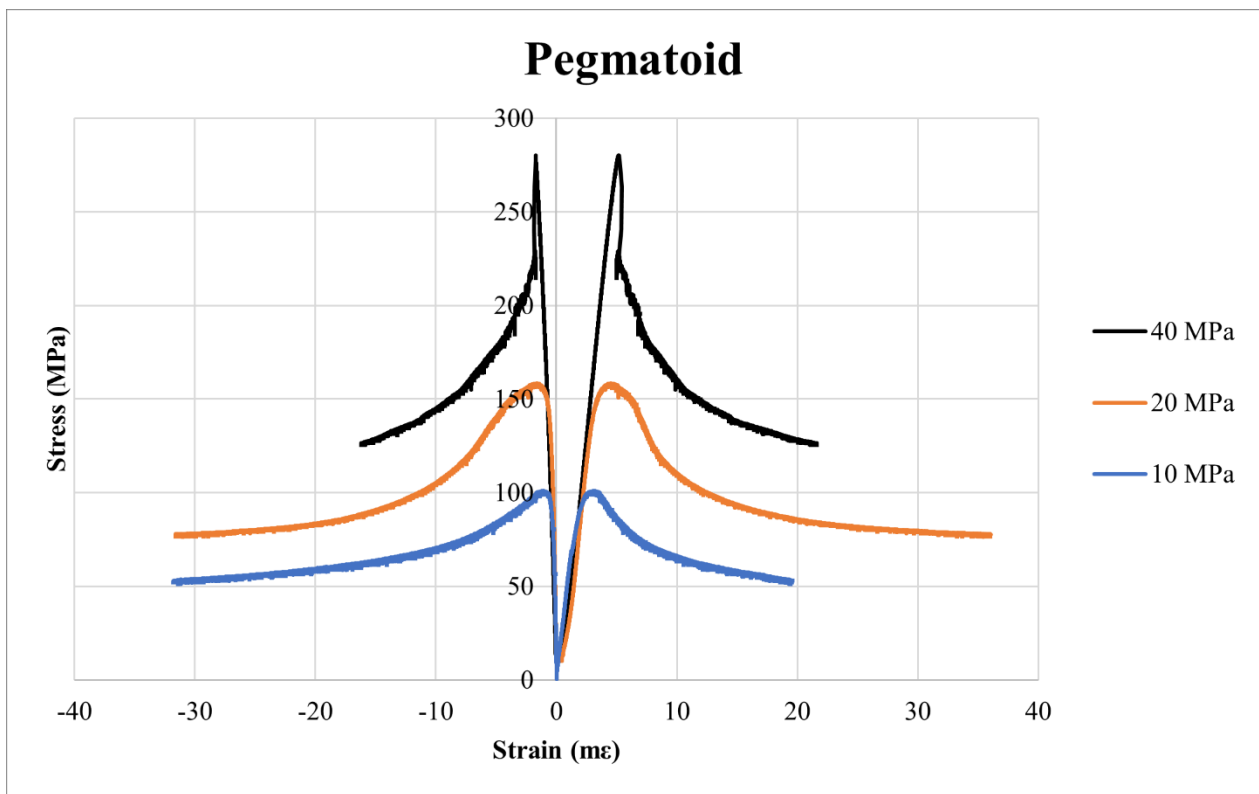


Figure 3.35 Stress-strain curves for pegmatoid

3.4.1.3 Chromitite results

Figure 3.36 shows the first set of stress-strain results at confinement levels of 10 MPa, 20 MPa and 40 MPa. The other sets of curves are shown in Appendix A (from Figure A-6 to A-8). Although all chromitite samples used for testing were obtained at depths of 1200

m and greater, the stress-strain curves at 10 MPa and 20 MPa resembled class I behaviour. Stress-strain curves obtained at 40 MPa for chromitite results sets 1 (Figure 3.36) and 3 (Figure A-7) suggest a transitional phase for the rock type. However, the results of set 2 for chromitite showed class I behaviour at 40 MPa.

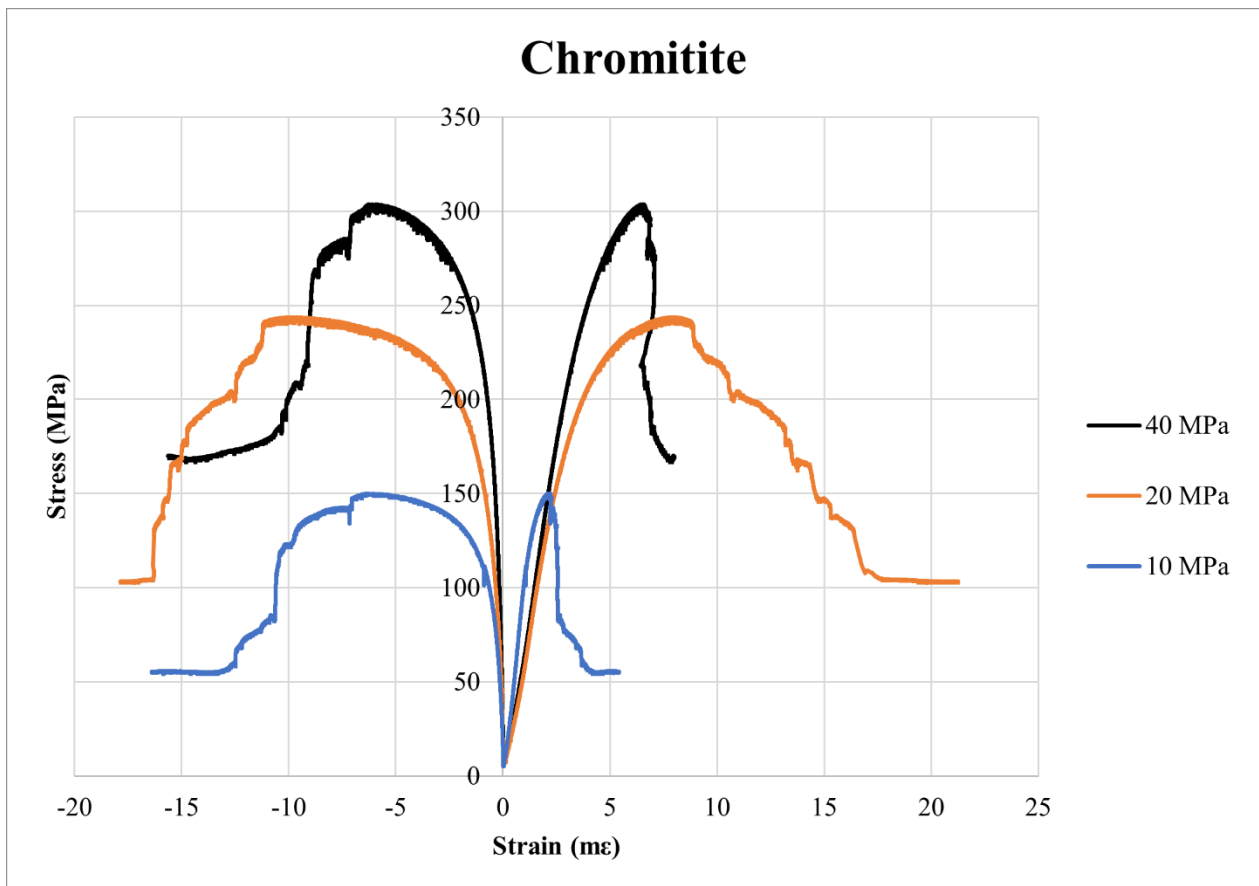


Figure 3.36 Stress-strain curves for chromitite

3.4.1.4 Anorthosite results

Anorthosite is known as a class II rock type (Watson, 2010). The specimens used in the triaxial tests for anorthosite were all obtained at depths greater than 1000 m and the results were consistent with Watson (2010)'s observation. At confinements of 10 MPa and 20 MPa, the stress-strain curves showed either a class I/class II (transition phase) or class

I behaviour. At 30 MPa all results (appendix A-8 to A-10) showed class I characteristics. Figure 3.37 shows the stress-strain curves for anorthosite obtained at confinements of 10 MPa, 20 MPa and 30 MPa.

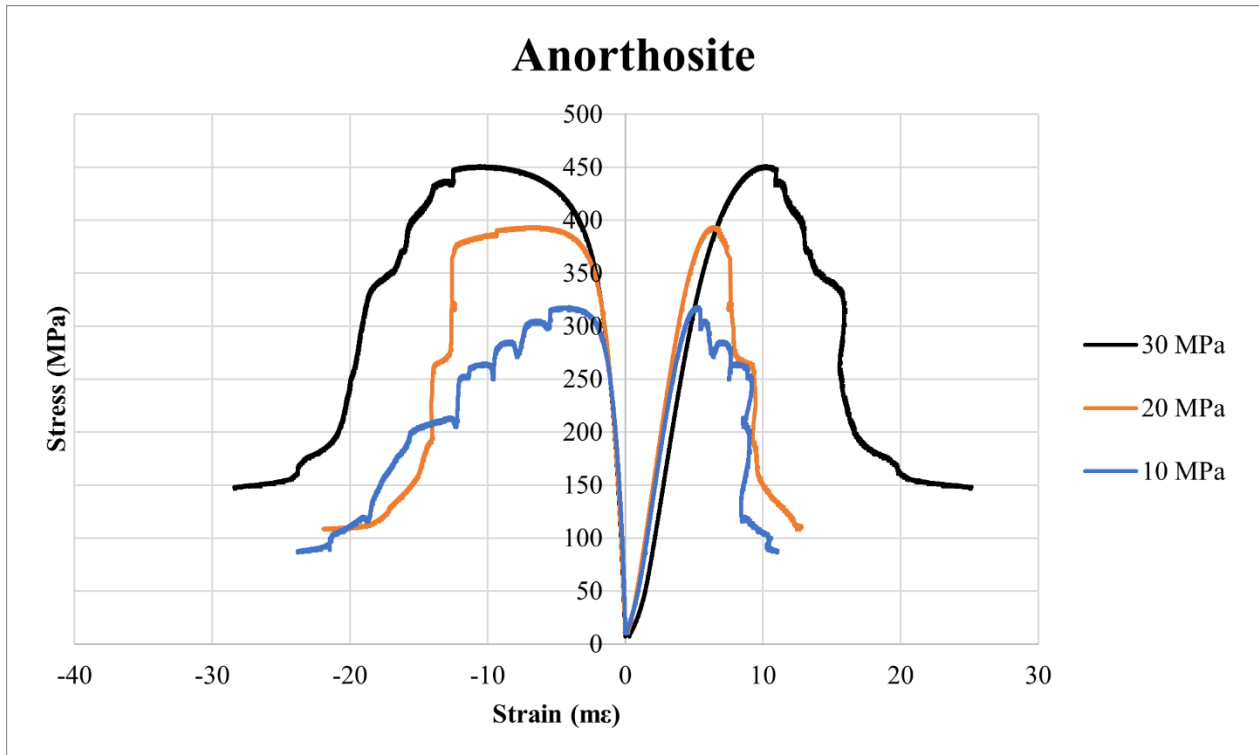


Figure 3.37 Stress-strain curves for anorthosite

3.4.1.5 The effect of confining stress

The effect of confinement on strength has been one of the most thoroughly researched areas and it has well been established that rock specimens tend to strengthen with increasing confinement. The same effect is observed on the residual strength as shown in Figure 3.38. In the range of confinements used in this research, little to no effect was observed on the Poisson's ratio. Table A-1 (Appendix A) shows the elastic constants at various confinements.

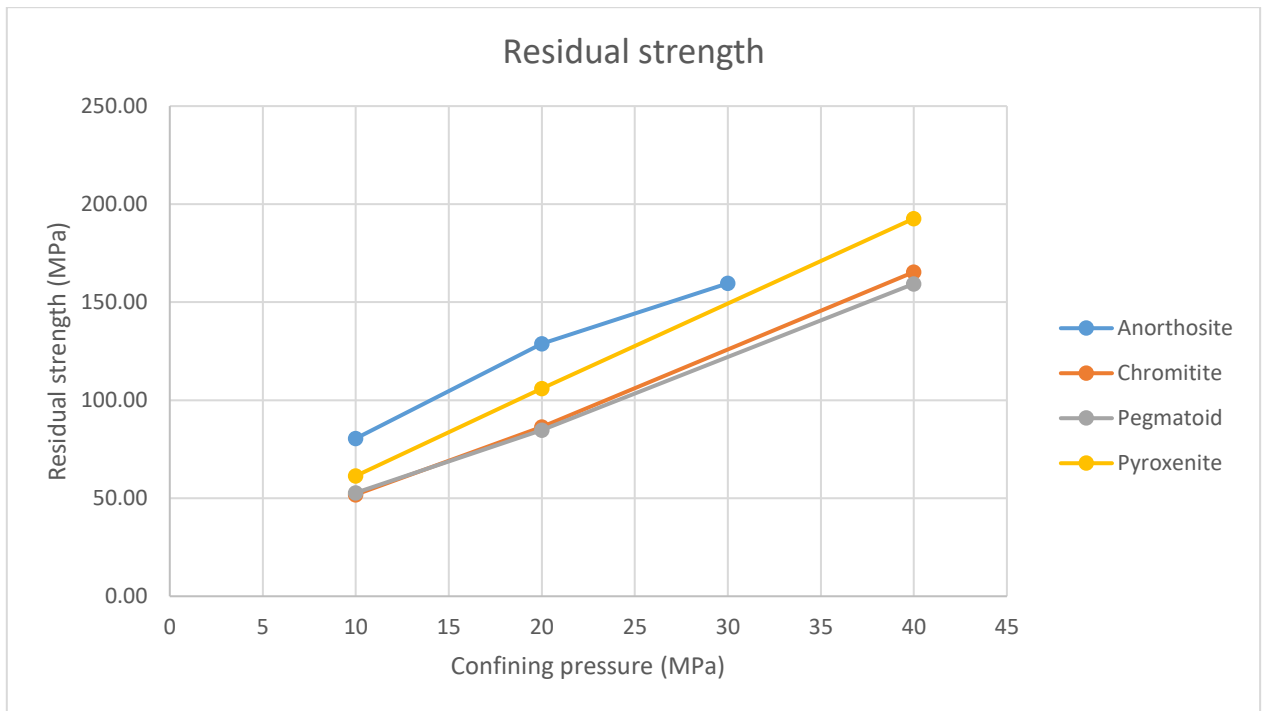


Figure 3.38 Effect confinement on residual strength

Effect of confinement on dilation

The dilation angle has been shown to be sensitive to confining pressure and it varies depending on what stage the rock specimen is at during triaxial testing. A set of three dilation angles (pre-peak, post-peak and residual dilation angles) were determined for each rock type. Plots of volumetric strain against axial strain were used for this purpose. For illustrative purposes, Figure 3.39 is given as an example on how to determine a set of dilation angles. The figure shows a plot of volumetric strain vs axial strain for a pyroxenite sample at 20 MPa confinement. The highlighted regions represent the pre-peak, post-peak and residual regions as shown in the legend. The gradient for the trend line for each region is used in Equation 2-4 to obtain the dilation angle. This gives the pre-peak, post-peak and residual dilation angles of 23.24° , 27.57° and 12.39° , respectively.

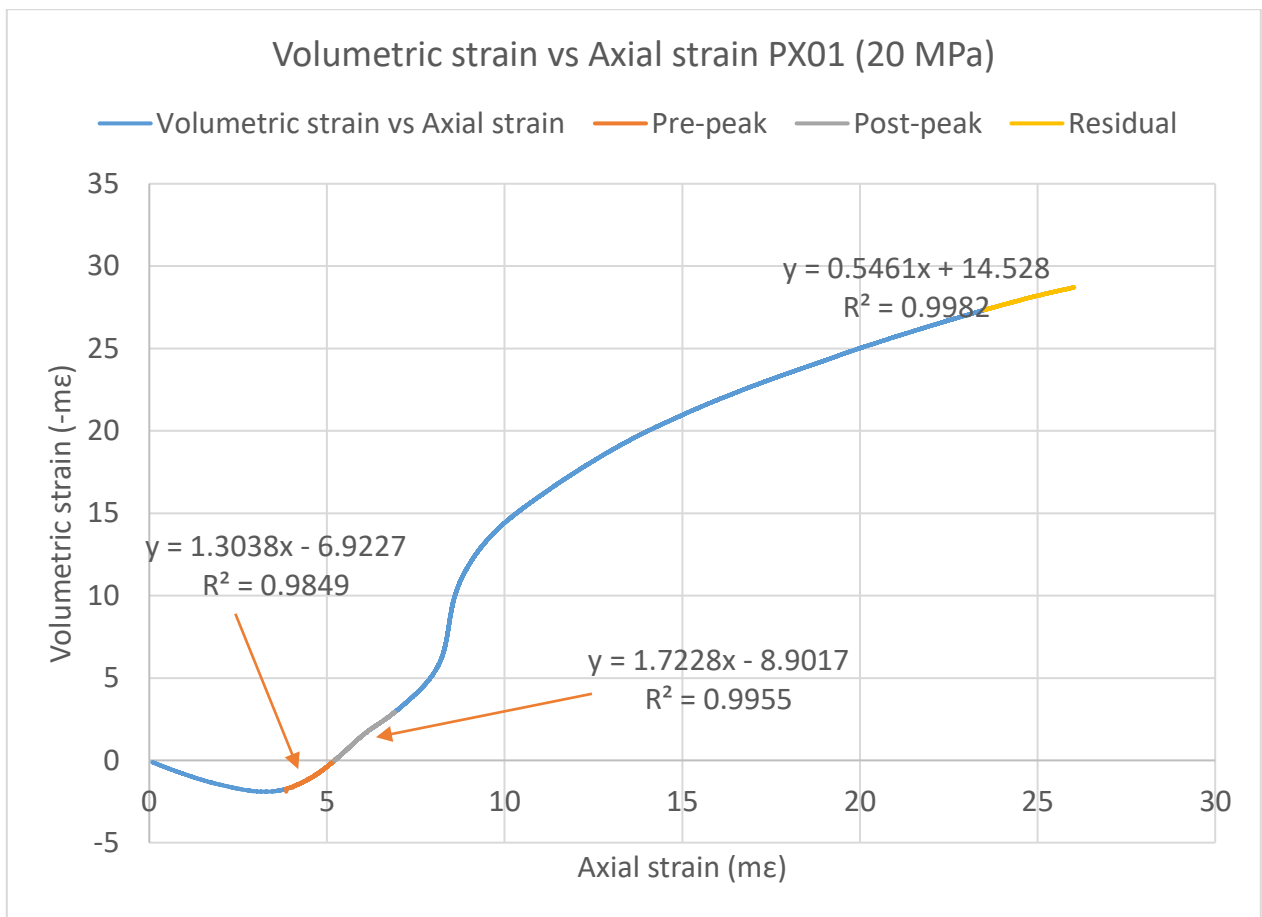


Figure 3.39 Plot of volumetric strain vs axial strain for a pyroxenite sample at 20 MPa confinement

The overall trend observed for dilation angle is that it tends to decrease with increasing confining pressure at all stages of rock deformation. This is shown in Figure 3.40 for pyroxenite. The dilation angles are highest in the post-peak phase followed by the pre-peak phase. The residual dilation angles are considerably smaller than the post-peak and the pre-peak dilation angles. This could be caused by more cracks that are interlinked at the residual stage, while at pre-peak and post-peak stage; more cracks are generated because of increasing axial load. The load results in the weakening of the bonds within the grains of the rock samples and contributes to higher volumetric expansion. Figure 3.40 shows the effect of confinement on dilation angle for pyroxenite. Similar trends are

observed in the other rock types. Plots of dilation angle vs confinement for the other rock types are shown in Appendix B (B-1 to B-3).

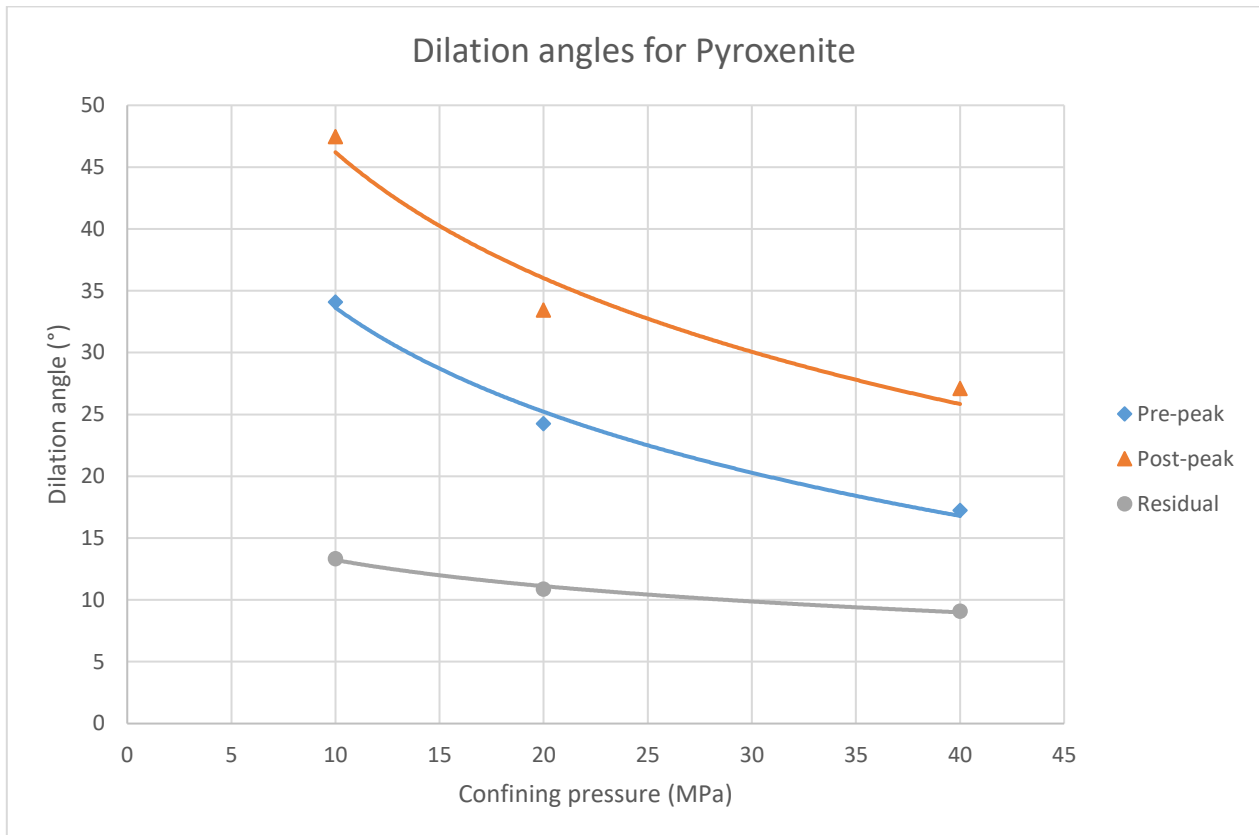


Figure 3.40 Plot of dilation angles vs confining pressure for pyroxenite

Zhang and Li (2019) performed triaxial tests on granite and the stress-strain curves they obtained are shown in Figure 3.41. In the post-peak stage, reversing loops are observed. Similar behaviour was observed on anorthosite specimens (Figures 3.37 A-9 and A-10) and a pegmatoid specimen (Figure A-5). Zhang and Li (2019) suggested that these loops are evidence of shear ruptures and that it is a progressive and stepwise process. However, observations made in this research suggest that the reversing loops maybe an indication that the radial strain control was set at a strain rate slightly higher than the rate ideal for that rock type. Reversing loops in pegmatoid disappeared when the radial strain

was lowered. The ideal radial strain rate was determined by trial and error. Unfortunately, there were insufficient anorthosite samples to test this hypothesis on this rock type. It was also noted that these loops are mostly observed in rocks that are very brittle, typically class II. It is thought that the loops may be the result of machine instability, briefly losing control and the loop is formed as the machine attempts to restore control. The loops were also observed to disappear at higher confinements which was also noted by Zhang and Li (2019).

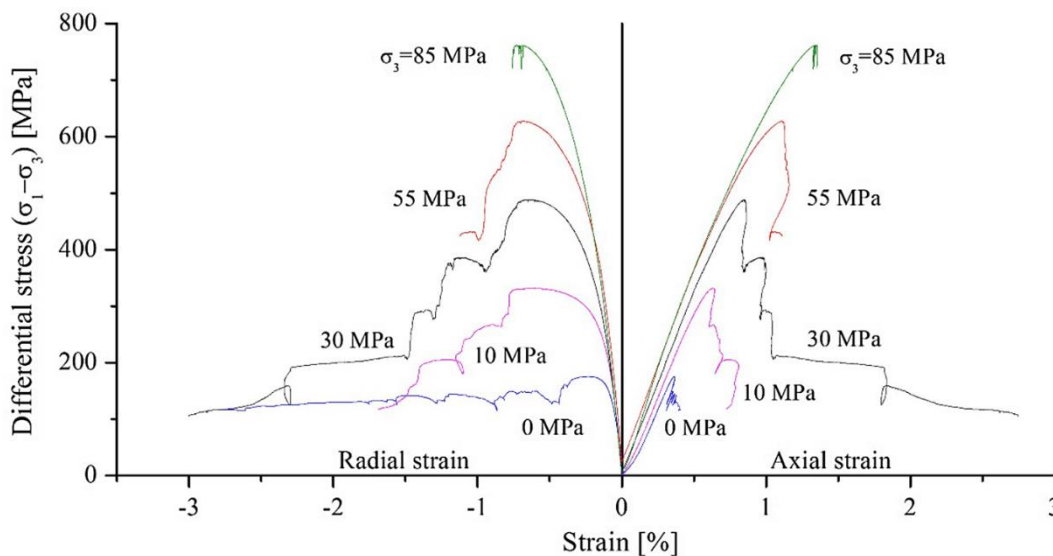


Figure 3.41 Stress-strain curves for granite (Zhang and Li, 2019)

Effect of confinement on the shear plane

In a triaxial test the specimen failure mode is shear and the angle at which the shear plane is formed is influenced by confinement (Tarasov and Potvin, 2013). This angle can be predicted using Mohr's circles and Mohr's failure envelope. The fracture angle (β) is defined as the angle between the vertical and the normal forces acting on the fracture

plane. This is illustrated in Figure 3.42. Tarasov and Potvin's (2013) hypothesis was tested as part of the research in this project. Mohr's non-linear failure envelope was also compared to the linear Navier-Coulomb envelope.

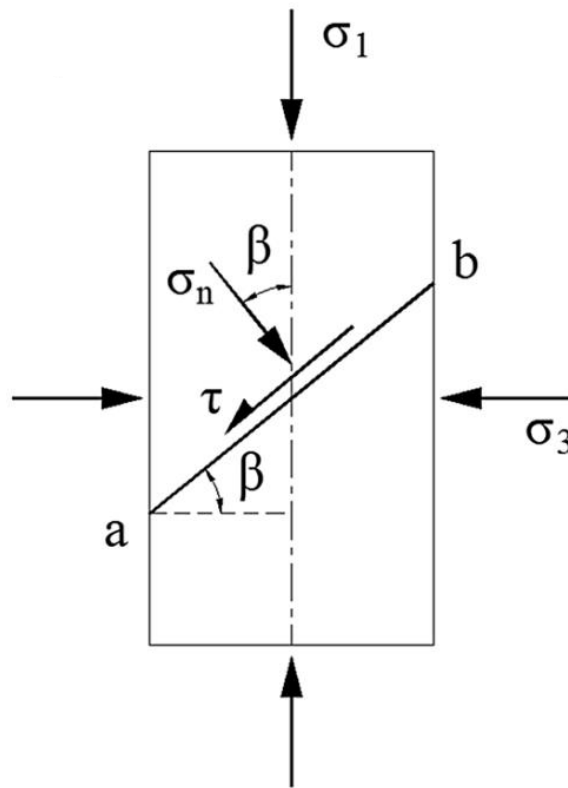


Figure 3.42 Illustration showing the fracture angle (β) (Zhang and Li, 2019)

The fracture angle (β) was measured for each rock type at different confinements. Figure 3.43 shows pairs of rock samples for each rock type and their respective failure angles. On the top left corner is the pyroxenite pair followed by the pegmatoid pair and the chromitite pair. At the bottom row is the anorthosite pair. For each pair, the samples on the left were tested at 10 MPa whilst the other sample was tested at 40 MPa except for the anorthosite pair, where the sample on the right was tested at 30 MPa.



Figure 3.43 The failure angles for each rock type

The observed trend is that β decreases with increasing confinement regardless of rock type. This result agrees with observations made by Tarasov and Potvin (2013) and Zhang and Li (2019). Mohr's failure criterion was used to predict the fracture angle at confinements levels used in this research. Figure 3.44 shows Mohr's failure envelope for pyroxenite. The Navier-Coulomb failure envelope is also included. The failure envelopes of the other rock types are shown in Appendix C (C-1 to C-3).

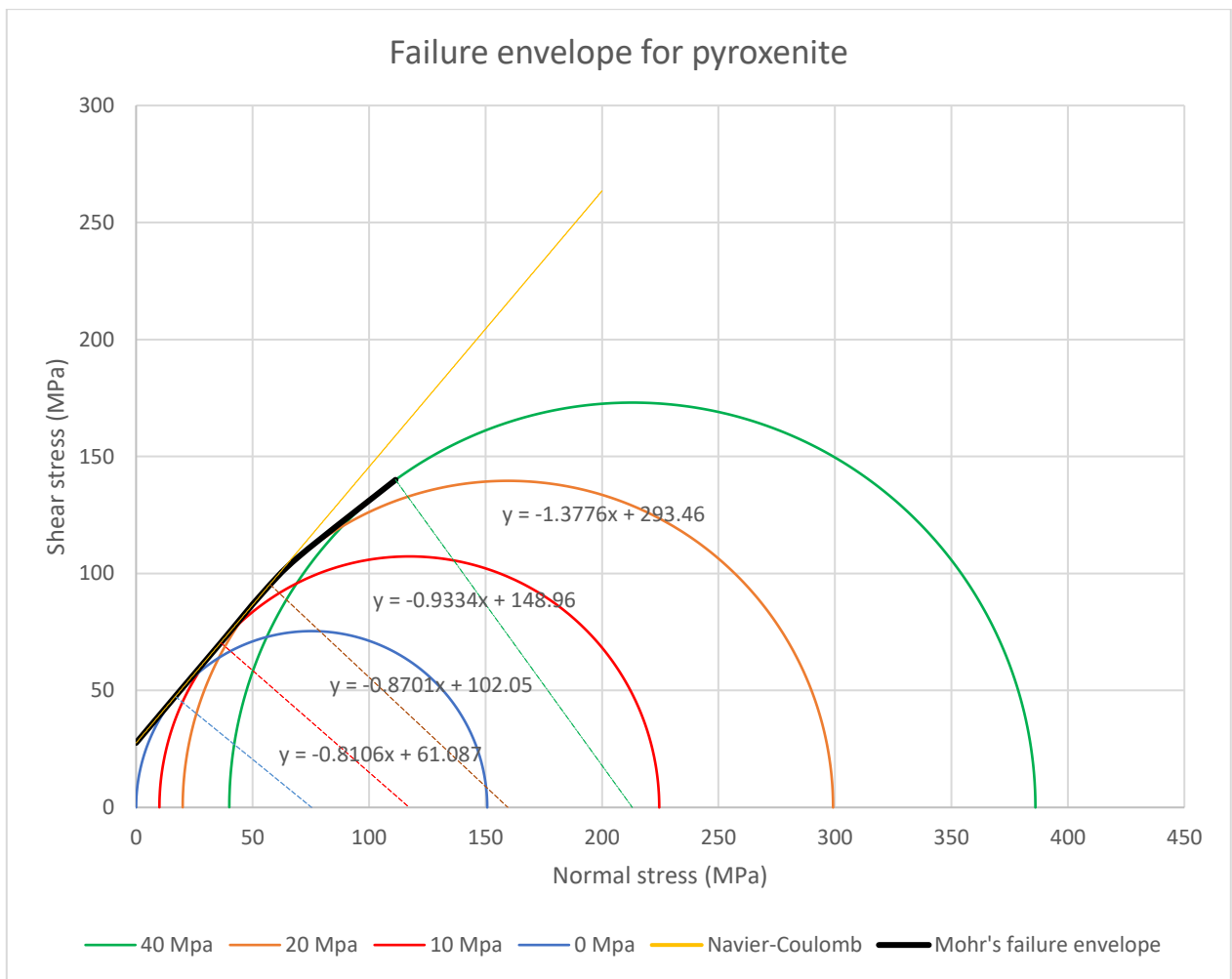


Figure 3.44 Failure envelopes for pyroxenite

Mohr's envelope was drawn tangent to each circle. The radii are drawn at the point where Mohr's envelope touches each circle. The Navier-Coulomb envelope was drawn using the line of best fit approach. The gradient of the radius of each Mohr circle was used to predict the fracture angle β at each confinement level. The angles in Figure 3.44 are 2β . A comparison between Mohr's envelope and the Navier-Coulomb envelope, shows that at confinement levels of 20 MPa and lower, the Navier-Coulomb envelope is a close approximation of Mohr's failure envelope. This trend was observed in all rock types used in this research. Table 3.3 compares the measured and predicted fracture angles obtained

for all confinements of each rock type. Overall, there is good agreement between the measured and the predicted angles. Predicted chromitite angles were almost identical to the measured angles. The rest of the measured angles are within $\pm 5^\circ$ of the predicted values except for pegmatoid at 40 MPa. The observations made suggest that Mohr's failure envelope can be used to predict the fracture angles of rock within the confinement range used in this research.

Table 3.3 Comparison fracture angles

	Confinement					
	10 MPa		20 MPa		40 MPa (30 MPa for anorthosite)	
Rock type	Measured (β)	Predicted (β)	Measured (β)	Predicted (β)	Measured (β)	Predicted (β)
Anorthosite	68°	72°	67°	71°	65°	70°
Pegmatoid	63°	68°	62°	67°	58°	66°
Pyroxenite	74°	69°	66°	68°	61°	63°
Chromitite	70°	71°	67°	70°	61°	61°

The formation of two or more fracture planes was observed on some of the tested samples with the most common case being two conjugate fracture planes. In the case of two fracture planes, it was observed that the fracture angles of the two shear planes were approximately the same (Figure 3.45). In general, the orientation of the fracture planes seems to be influenced by the confinement.

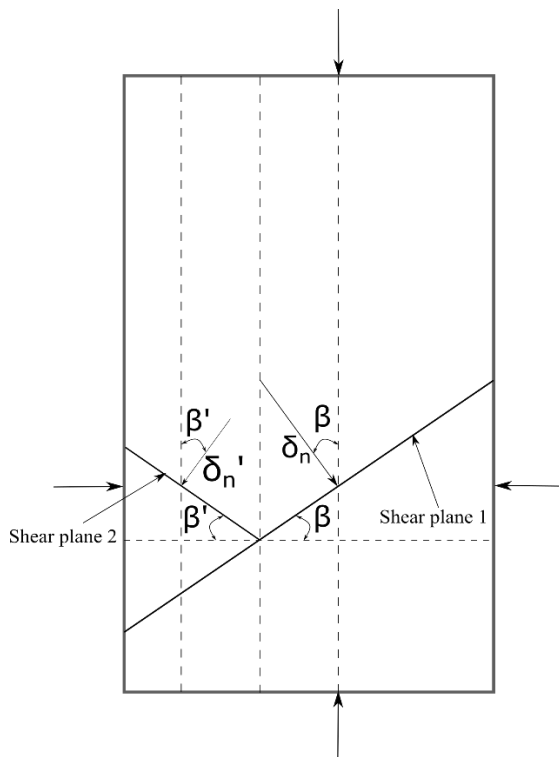


Figure 3.45 Diagram showing the fracture angles of the two fracture planes

The case of two shear planes was observed in all the rock types except for chromitite. It is important to point out that the case of two shear planes is a rare event. For the 100 triaxial tests conducted, only four exhibited two shear planes. The formation of two shear planes was only observed at confinements of 30 MPa or higher. Figure 3.46 shows rock samples with two fracture planes. The sample on the left is pegmatoid (40 MPa) followed by two pyroxenite samples (40 MPa) and anorthosite (30 MPa) on the right.

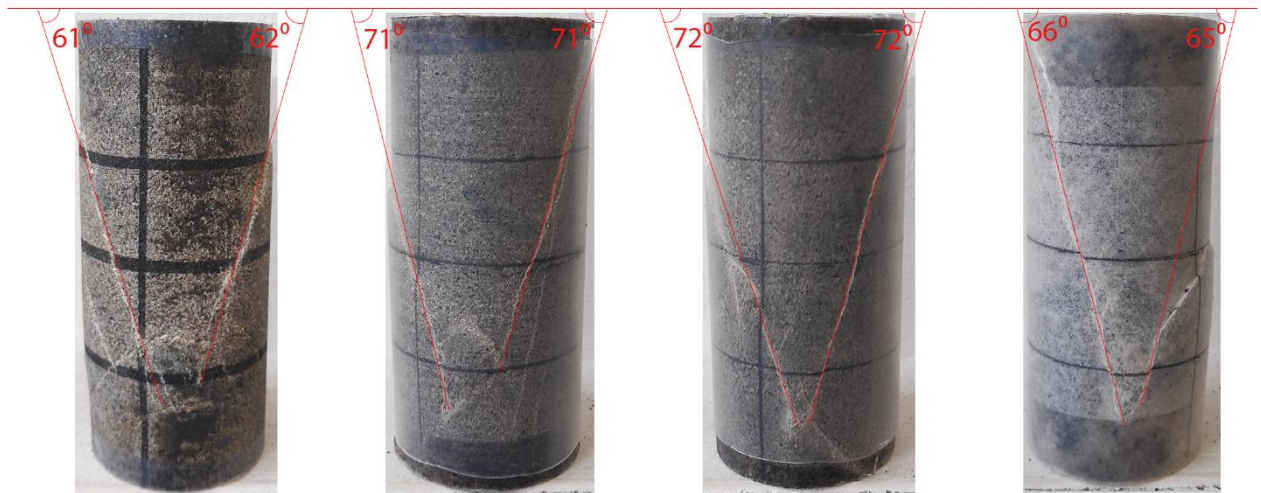


Figure 3.46 Fracture angles for samples having two shear planes

The angles predicted by Mohr's failure criterion were close for pegmatoid and anorthosite, however a significant difference is noted for the pyroxenite samples. In one instance, multiple shear planes were observed in a pyroxenite sample (Figure 3.47).



Figure 3.47 Sample having multiple shear fractures

The reason behind the formation of multiple shear fractures is unknown, but it is suspected to be due to pre-existing flaws in the rock. The sample's peak strength was only about 5% lower than the average strength. The difference in strength is not significant enough to conclude that the sample had pre-existing fractures. Interestingly, the fracture planes formed in the specimen are almost parallel to each other. From an energy perspective the shear plane is formed from the accumulation of elastic energy at peak strength. The release of this energy causes shear ruptures resulting in localized failure. In the scenario of multiple shear planes, it may be assumed that multiple shear ruptures occurred in the paths of least resistance.

3.4.2 Data comparison

The data processed in this chapter was obtained from two different sources, namely the laboratory (Wits) tests and from the mine's laboratory. It is important to check the quality of data provided by the mine as it impacts the results produced by the numerical model. The expected outcome is that the data from the two sources should show comparable

trends, as the data was obtained from the same type of rocks from the same mine. The data provided by the mine contains results from triaxial, UCM and Brazilian tests. The same tests were done in the laboratory with the only difference being post-peak behaviour in the triaxial tests.

3.4.2.1 Brazilian test results

Table 3.4 shows the results of the Brazilian tests from the two sources. The tensile strength for chromitite is almost the same for the two sources. The tensile strengths for the other rock types are within ± 3 MPa of each other. The table also shows the standard deviation for each rock type. The mine data has higher standard deviation as compared to the lab tests because of the bigger sample population. The average tensile strength was calculated using Equation 3.4.

Table 3.4 Brazilian test results

	Lab data		Mine data		σ_t
Rock type	Tensile strength (MPa)	Standard deviation	Tensile strength (MPa)	Standard deviation	Average tensile strength (MPa)
Anorthosite	5.29	0.60	8.55	2.55	8.19
Chromitite	3.75	0.62	3.53	2.31	3.57
Pegmatoid	3.39	0.66	5.76	1.46	5.23
Pyroxenite	5.80	0.56	8.12	3.11	7.82

$$\sigma_t = \frac{m \times \sigma_m + n \times \sigma_n}{m + n} \quad (3-4)$$

Where σ_t is the weighted average tensile strength,
 σ_m is the mean tensile strength from the mine data,
 σ_n is the mean tensile strength from the lab test,
m is the number of samples tested at the mine,
n is the number of samples tested in the laboratory.

3.4.2.2 UCS and triaxial test results

The peak strengths from the UCS and triaxial tests were plotted in the δ_1 - δ_3 space (peak strength vs confining stress) as shown in Figure 3.48. The solid lines represent the failure envelopes obtain from lab tests and the dashed lines represent envelopes obtained from the mine data. The failure envelope for each rock type is assigned a colour:

- Blue represents chromitite;
- green represents anorthosite;
- red represents pyroxenite; and
- black represents pegmatoid.

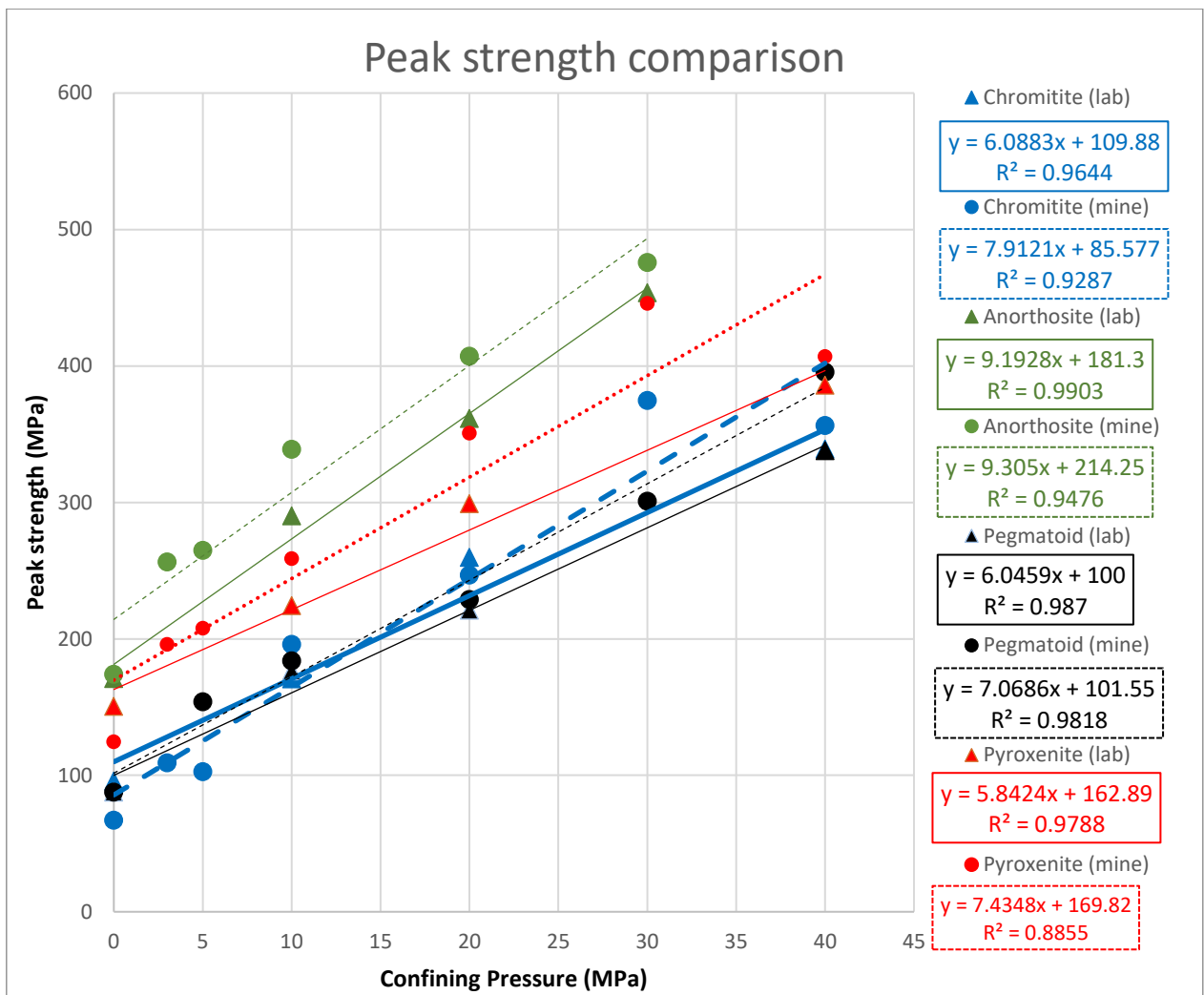


Figure 3.48 Plot of peak strength vs confining Pressure

The two failure envelopes for anorthosite are almost parallel. The peak strengths from the mine data are higher than those obtained in the laboratory. The UCS values from the two sources are very close. The rest of the peak strengths at each confinement are within ± 10 MPa of each other, which is acceptable. The failure envelopes for the remaining rock types diverge with increasing confinement. Pyroxenite has the greatest divergence with the difference between peak strength at each confinement in the range 0.1 MPa – 50 MPa. Chromitite and pegmatoid failure envelopes have similar divergence and the differences between each peak strength are within ± 5 MPa. The differences between the

results could be caused by several factors. The storage of the rock cores can cause differences in results obtained.

Table 3.5 shows the internal friction angle and the cohesion for each rock type. Figure 3.48 together with Equations 3-5 and 3-6 were used to determine these for each data set. The obtained internal friction angles are within 3° of each other and the differences in cohesion are within 4 MPa. Overall, the two sources provided results with similar trends. The average values of the obtained internal friction angles and cohesion from the two sources were taken. The average values were used in the numerical model described in Chapter 4.

Table 3.5 Values of cohesion and internal friction angle.

	Lab data			Mine data			Average	
	m	Φ	C (MPa)	m	Φ	C (MPa)	Φ	C (MPa)
Anorthosite	9.1928	53.49	28.22	9.305	53.70	28.56	53.60	28.39
Chromitite	6.0883	45.88	19.41	7.9121	50.86	11.93	48.37	15.67
Pegmatoid	6.0459	45.74	17.85	7.0686	48.77	16.53	47.26	17.19
Pyroxenite	5.8424	45.05	31.18	7.4348	49.72	22.87	47.38	27.02

$$\phi = \sin^{-1} \left(\frac{m-1}{m+1} \right) \sigma_c \quad (3-5)$$

$$C = \left(\frac{1 - \sin \phi}{\cos \phi} \right) \sigma_c \quad (3-6)$$

Where σ_c is the UCS of the rock type,

m is the strengthening parameter (gradient) shown in Figure 3.48,

c is the cohesion of intact rock,

Φ is the internal friction angle.

The elastic moduli and the Poisson's ratios obtained from UCM tests are shown in Table 3.6. The Poisson's ratios obtained from the two Anorthosite sets are almost the same however, the elastic moduli are significantly different. The Young's moduli and Poisson's Ratios were consistently higher and lower, respectively, in the Lab samples, for all rock types. Generally, the Young's moduli were significantly higher in the Lab tests, when compared to the Mine samples. The mean of the elastic constants was taken. If the difference between the Poisson's ratios is greater than 0.05, the lab determined values were used.

Table 3.6 Comparison of elastic constants

	Lab		Mine		Average	
Rock type	Elastic Modulus (GPa)	Poisson's ratio	Elastic Modulus (GPa)	Poisson's ratio	Elastic Modulus (GPa)	Poisson's ratio
Anorthosite	94.39	0.42	64.20	0.43	79.29	0.43
Chromitite	89.05	0.33	58.90	0.41	73.98	0.33
Pegmatoid	57.42	0.23	56.90	0.25	57.16	0.24
Pyroxenite	148.27	0.23	64.05	0.34	106.16	0.23

The bulk and shear moduli for each rock type were calculated from the average results shown in Table 3.6 using Equations 3-7 and 3-8. Table 3.7 shows the bulk and shear modulus values obtained for each rock type.

Table 3.7 Bulk and shear modulus values

Rock type	Bulk modulus (GPa)	Shear modulus (GPa)
Anorthosite	176.21	27.82
Chromitite	72.53	27.81
Pegmatoid	36.64	23.05
Pyroxenite	65.53	43.15

$$\text{Bulk Modulus } (K) = \frac{E}{3(1 - 2\nu)} \quad (3-7)$$

$$\text{Shear Modulus } (G) = \frac{E}{2(1 + \nu)} \quad (3-8)$$

Where E is the Axial tangential Young's modulus at 50% of UCS,
 ν is the Poisson's ratio at 50% of UCS.

3.5 Chapter summary

The first part of this chapter described the challenges that were faced during the data collection phase of the research and the solutions to these problems. The methods and equipment used in this research are also described in this section. Issues with the axial transducer were discovered during the testing phase. Investigations ultimately led to the replacement of the axial transducer. Triaxial tests were conducted at confinements of

10 MPa, 20 MPa and 40 MPa except for the anorthosite samples that were tested at a confinement of 30 MPa instead of 40 MPa.

The second part describes the laboratory results. A comparison between the laboratory results and the database, provided by the mine, showed close agreement and the means of the data sets were used to determine the cohesion and the internal friction angles for each rock type. Anorthosite and pyroxenite both exhibited transitional behaviour at confinements of 10 MPa and 20 MPa, and both rock types exhibited class I behaviour at confinements greater than 20 MPa. Pegmatoid exhibited class I behaviour at confinements of 10 MPa and 20 MPa, and transitional behaviour at 40 MPa. Chromitite exhibited behaviour similar to the pegmatoid with a difference noted at 40 MPa where some of the curves obtained suggested class I behaviour. The dilation angle was observed to decrease with increasing confinement. This could be caused by the increasing pressure provided by the confining fluid restricting the ability of the specimen to dilate. Mohr's failure criterion was shown to be a good predictor of the fracture angle in triaxial tests. It was also observed that the fracture angle tends to decrease with increasing confinement. In the case of two conjugate shear planes being formed it was observed that the fracture angles of the two planes are the same.

4 NUMERICAL MODELLING

This chapter presents the FLAC3D numerical modelling of a 4 m x 2 m crush pillar. The aim of the modelling was to determine the stress conditions required for the crush pillar to work effectively. The process required FLAC3D simulations of the laboratory triaxial tests, attempting to reproduce the stress-strain curves obtained in the laboratory. The input parameters for the numerical models were obtained from the results shown in Chapter 3. Subsequently the numerical simulations were down-rated to account for the rock mass. The data files for all models are presented in Appendix D.

4.1 FLAC3D model input parameters

The mechanical parameters determined in Chapter 3 are summarized in Table 4.1. The parameters were used in triaxial models. The objective of these models was to try and reproduce the stress-strain curves obtained in the laboratory experiments. The dilation angles shown in Table 4.1 were for the 10 MPa confinement results.

Table 4.1 Numerical model input parameters

	ψ			ϕ	C	Tension
	Pre-peak	Post-peak	Residual		(MPa)	(MPa)
Anorthosite	32.21	46.98	23.94	53.60	28.39	8.19
Chromitite	19.91	32.82	10.56	48.37	15.67	3.57
Pegmatoid	31.12	38.43	10.86	47.26	17.91	5.23
Pyroxenite	34.10	47.49	13.34	47.38	27.02	7.82

4.1.1 The plastic shear strain parameter

In addition to the values in Table 4.1, FLAC3D requires a failure criterion and the Mohr–Coulomb strain-hardening/ softening model was used. This criterion requires a plastic shear strain parameter and the residual values of cohesion and internal friction angle. The cohesion, internal friction angle and dilation angle were expressed in terms of the shear parameter in the form of a table. The shear strain parameter (ϵ^{ps}) was determined from the stress-strain curve. Figure 4.1 shows a stress/strain curve for chromitite.

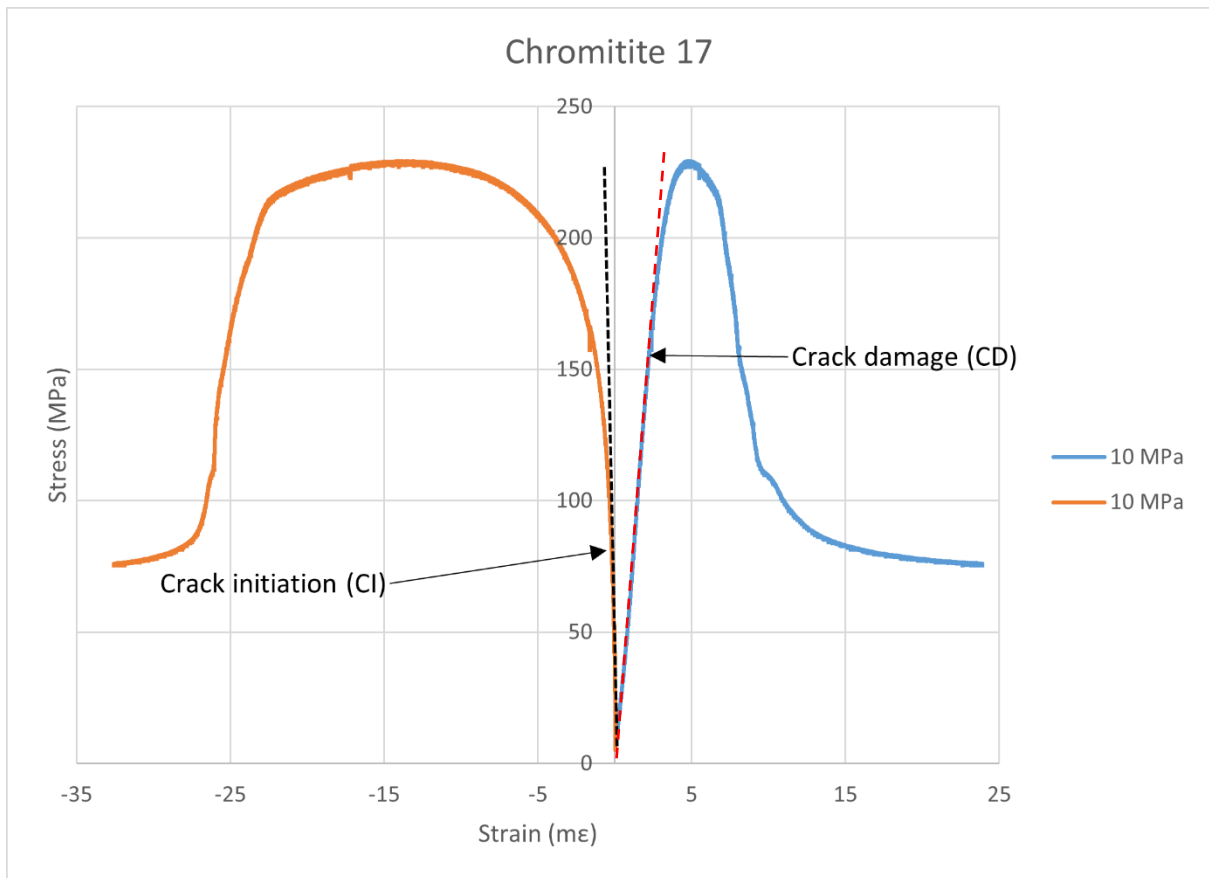


Figure 4.1 Stress-strain curve for a chromitite sample with points of crack initiation and crack damage marked

The crack initiation and crack damage stresses (Figure 4.1) are crucial in determining the shear strain parameter (ϵ^{ps}). Crack initiation (CI) stress refers to the stress level that coincides with the onset of dilation (Zhao and Cai, 2010). This point is also the onset of stable cracking (Walton and Diederichs, 2015). Crack damage (CD) is defined by Walton and Diederichs (2015) as the point of non-linearity on an axial stress-axial strain curve. Most authors describe this point as the onset of unstable crack propagation (Cai et al., 2004; Kotsanis et al., 2021; Oniyide, 2015; Zhao and Cai, 2010). The strains corresponding to these stress points are also important.

To determine ϵ^{ps} the CI and CD and the corresponding $\epsilon_{1(CI)}$ and $\epsilon_{1(CD)}$ strain values were used in Equations 4-1 to 4-3 to determine the plastic strain (Walton and Diederichs, 2015).

$$\epsilon_1^p = \epsilon_1 - \epsilon_{1(CD)} - \frac{\sigma_1 - CD}{E} \quad (4-1)$$

$$\epsilon_3^p = \epsilon_3 - \epsilon_{3(CI)} + \nu \cdot \frac{\sigma_1 - CI}{E} \quad (4-2)$$

$$\epsilon_v^p = \epsilon_1^p + 2 \cdot \epsilon_3^p \quad (4-3)$$

Where E = Young's modulus

ν = Poison's ratio

ϵ_1^p = Axial plastic strain

ϵ_3^p = Radial plastic strain

ϵ_v^p = Volumetric plastic strain

σ_1 = Axial stress

CI = Crack initiation stress

CD = Crack damage stress

$\epsilon_{1(CI)}$ = Crack initiation strain

$\epsilon_{1(CD)}$ = Crack damage strain

The next step was to determine the plastic shear strain using Equation 4-4 (Walton and Diederichs, 2015).

$$\gamma^p = \varepsilon_1^p - \varepsilon_3^p \quad (4-4)$$

Where γ^p = is the plastic shear strain

ε_1^p = Axial plastic strain

ε_3^p = Radial plastic strain

The plastic shear strain is then adjusted to satisfy the condition of yield. Zhao and Cai (2010) suggested that the plastic shear strain must be adjusted to satisfy the condition $\gamma^p = 0$ at CD. This was done using Equation 4-5 (Walton and Diederichs, 2015).

$$\gamma_{Adjusted}^p = \gamma_{Original}^p - \gamma_{CD}^p \quad (4-5)$$

Equation 4-5 was used to determine the $\gamma_{Adjusted}^p$ for the pre-peak, post peak and residual regions. The $\gamma_{Adjusted}^p$ values together with their corresponding dilation angles (Table 4.1) are substituted in Equation 4.6 to obtain ε^{ps} values for the corresponding regions (Zhao and Cai, 2010).

$$\varepsilon^{ps} = \frac{\sqrt{3}}{3} \sqrt{1 + N_\psi + N_\psi^2} \frac{\gamma^p}{1 + N_\psi} \quad (4-6)$$

Where $N_\psi = \frac{1+\sin \psi}{1-\sin \psi}$

γ^p = is the adjusted plastic shear strain

ε^{ps} = is the plastic shear strain parameter

The values of ϵ^{ps} obtained from stress-strain curves are presented in Table 4.2. The value of ϵ^{ps} for the pre-peak region is set to zero as it coincides with crack damage stress (CD) making

$$\gamma^p = 0.$$

Table 4.2 Shows the shear strain parameters used in the triaxial model

	Pre-peak	Post-peak	Residual
Anorthosite	0	0.002	0.02
Chromitite	0	0.0008	0.007
Pegmatoid	0	0.001	0.02
Pyroxenite	0	0.003	0.02

4.1.2 Residual cohesion and internal friction angle

The residual values of cohesion and internal friction angle were estimated using Mohr's circles plotted in the shear-normal stress space. Figure 4.2 shows an example for chromitite. The residual strength and the confinement are plotted on the normal stress axis. The residual values for each rock type are shown in Table 4.3. The residual cohesion for pyroxenite had to be adjusted as the model showed a low residual value compared to those obtained in Chapter 3. The estimated residual values are shown in Table 4.3

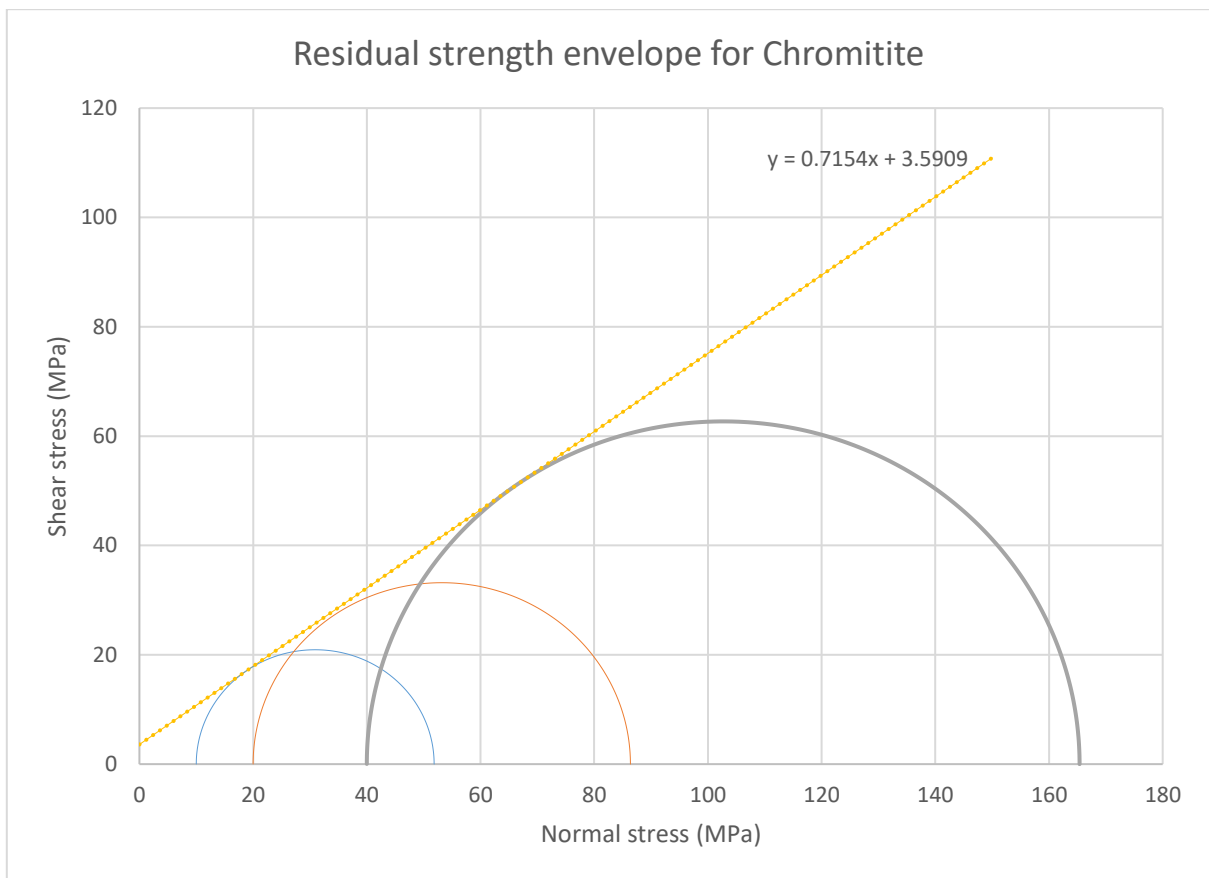


Figure 4.2 Estimation of residual cohesion and internal friction angle using Mohr's circles

Table 4.3 Residual values of cohesion and internal angle of friction

	Cohesion (MPa)	Internal friction angle
Anorthosite	10	36.87
Chromitite	3.59	35.58
Pegmatoid	2	35.58
Pyroxenite	13	38.9

4.2 Triaxial model

A cylinder model (representing rock samples tested in the laboratory) was setup in FLAC3D. The cylinder has a height of 117.5 mm and a diameter of 47 mm. These dimensions are close to those on the specimens used in laboratory tests. The strain softening model was selected and the parameters in Tables 4.1-4.3 were applied. The density, bulk and shear moduli were also applied. To simulate how the MTS machine loads the specimen, a roller boundary was applied at one end of the cylinder to mimic the bottom loading piston. The other end of the cylinder is fixed, mimicking the stationary platen. The curved sides of the cylinder are left free to allow the confinement stress to act on it once initiated. A confinement of 10 MPa is initialized, and a velocity of 10^{-6} m/s is applied at the bottom of the cylinder. Figure 4.3 shows the triaxial model setup.

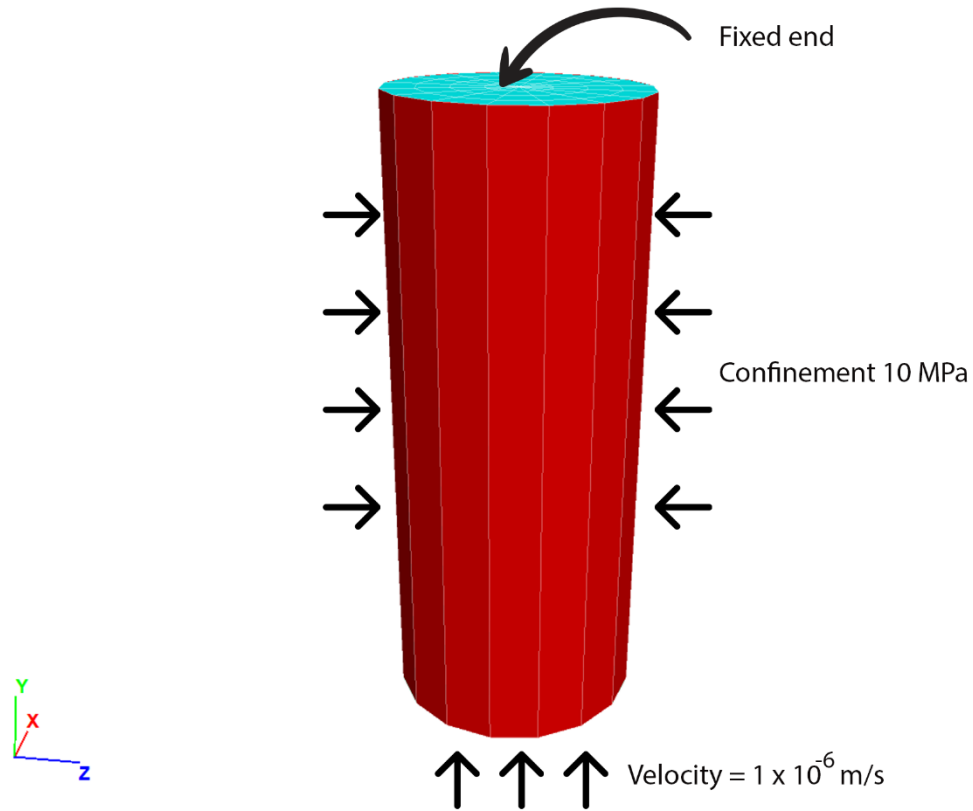


Figure 4.3 Model set up for triaxial simulation

The model was setup to use axial strain control. This means the model cannot simulate class II. Attempts to use radial strain control were unsuccessful. In general, the model underestimated the peak strength of the model samples. This could be caused by the values in Table 4.1 which are representative of the average values at 10 MPa rather than individual samples. The residual strength values from the model were close to those obtained from laboratory tests. The peak and residual strength values from the model and the laboratory results are shown in Table 4.4.

Table 4.4 Comparison of modelled values and lab test results

	Modelled values		Laboratory values	
	Peak strength (MPa)	Residual strength (MPa)	Peak strength (MPa)	Residual strength (MPa)
Anorthosite	236	84	277	86
Chromitite	110	45	124	43
Pegmatoid	124	53	101	52
Pyroxenite	184	55	198	65

The modelled stress-strain curves were similar to those obtained in Chapter 3. The differences were noted in areas where the curves from the laboratory tests exhibited class II. Figure 4.4 shows the simulated stress-strain curve for pyroxenite at 10 MPa. The simulated curves for the other rock types are shown in Appendix D.

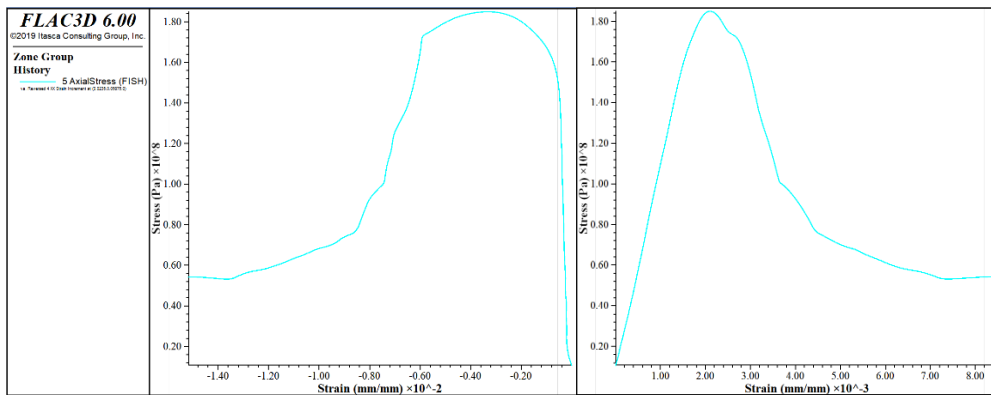


Figure 4.4 Simulated stress-strain curves for pyroxenite at 10 MPa

For comparison the laboratory determined stress-strain curve for pyroxenite at 10 MPa is reproduced in Figure 4.5. Overall, the model manages to reproduce similar shaped curves.

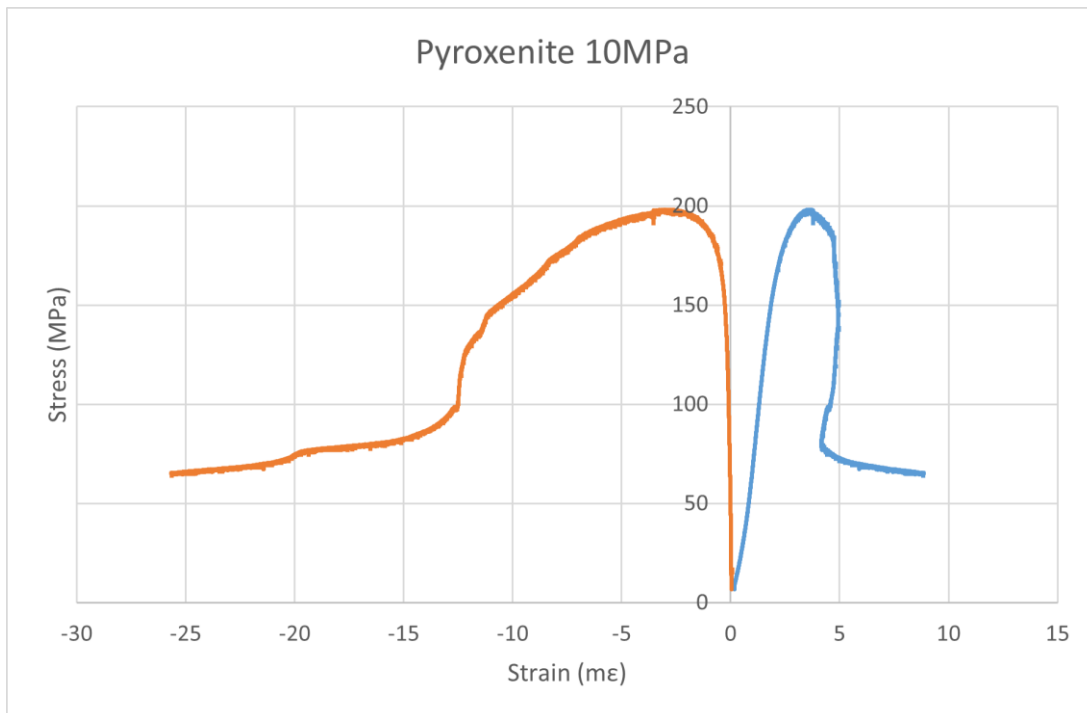


Figure 4.5 Stress-strain curve for pyroxenite at 10 MPa

4.3 Crush pillar model

4.3.1 Downrating of input parameters

The input parameters used in simulating crush pillar behaviour are similar to those used in the triaxial model. The parameters for the crush pillar model were determined from laboratory results and the data provided by the mine. The Rocscience software “RocLab” was used for downrating parameters and the inputs are presented in Table 4.5. The software uses Hoek-Brown criterion inputs to provide downrated Mohr-Coulomb parameters. The symbol (δ_{ci}) represents the compressive strength, (E_i) the intact Young’s modulus and (m_i) the material constant. A screen shot of the software is shown in Figure 4.6.

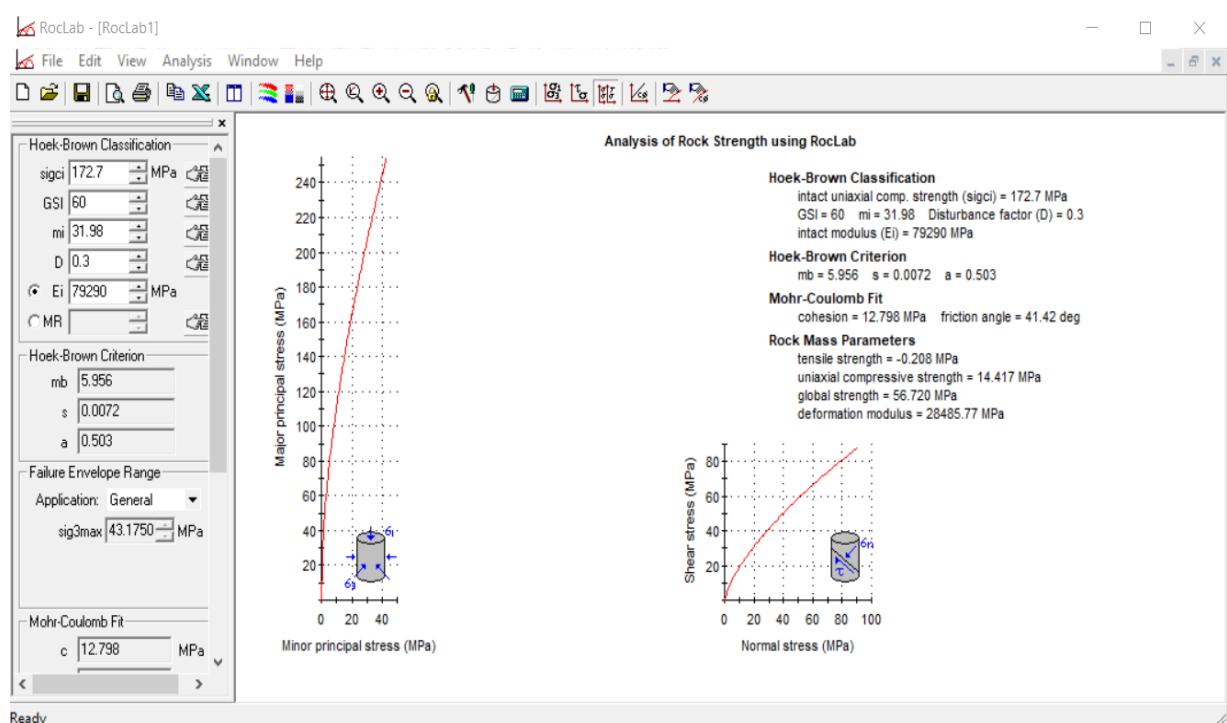


Figure 4.6 Screen shot of Roclab software used for downrating the input parameters

Table 4.5 Intact rock parameters

Rock type	δ_{ci} (MPa)	m_i	E_i (GPa)
Anorthosite	172.70	31.98	79.29
Chromitite	81.45	43.67	73.98
Pegmatoid	87.89	11.83	57.16
Pyroxenite	137.36	35.77	106.16

The disturbance factor (D) and the geological strength index (GSI) are also required by the software. D refers to the quality of blasting and ranges between 1 and 0, with 0 representing well controlled blasting (minimal damage to rock mass) and 1 representing poor blasting control (significant damage to rock mass) (Hoek et al., 2002). For platinum mines the D is assumed to vary between 0 and 0.5 (Oniyide, 2015). Issues of floor heave are prominent at D = 0.5. D in this investigation was set to 0.3, because it appears to be a reasonable approximation of underground observations. The GSI value was set to 60 (Gardner, 2022). The downrated Mohr-Coulomb parameters obtained using the described parameters in the Roclab software are presented in Table 4.6. The rock mass dilation angle was estimated by dividing the friction angle by 4 (Hoek and Brown, 1997).

Table 4.6 Crush pillar model input parameters. C is the cohesion, Φ is the friction angle, E_m is the down rated elastic modulus, δ_t is the tension, K is the bulk modulus, G is the shear modulus and ψ is the dilation angle.

Rock type	C (MPa)	Φ (°)	E_m (GPa)	δ_t (MPa)	K (GPa)	G (GPa)	ψ
Anorthosite	12.8	41.4	28.5	-0.208	67.86	9.97	10.4
Chromitite	6.6	44.1	26.6	-0.072	26.08	10	11
Pegmatoid	4.9	32.8	20.5	-0.28	13.14	8.27	8.2
Pyroxenite	10.5	42.39	38.1	-0.148	23.52	15.49	9.5

4.3.2 The influence of interfaces

Before creating the main (crush pillar) model, it was necessary to investigate the impact of lithological interfaces on crush pillar behaviour. This helped in deciding whether to include or exclude interfaces in the main model of the pillar. In this research, the term interface refers to the plane where two lithologies meet. The crush pillar modelled in this chapter comprises of three interfaces. A simple model of the crush pillar was set up in FLAC3D to investigate the effect of these interfaces on the behaviour of the pillar. The influence of draping and the presence of a gully were excluded. A quarter symmetry analysis was used. The pillar dimensions were 2 m x 1 m x 0.91 m (quarter symmetry).

The thickness of the hangingwall and footwall was set to 1 m (Roberts, 2022). The constitutive model was set to the Mohr-Coulomb (strain softening) criterion with non-associated shear flow rules. Roller boundaries were applied on the sides and base of the model as shown in Figure 4.7. A velocity of 10^{-6} m/s was applied at the top.

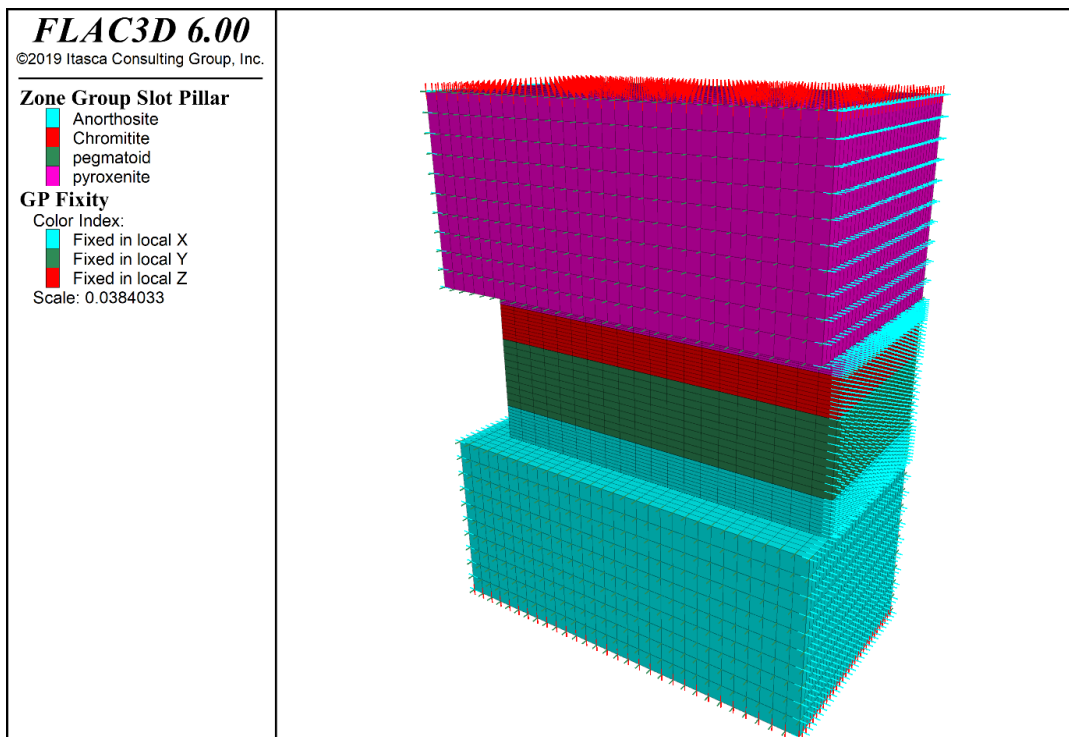


Figure 4.7 Boundary conditions applied to model

The interfaces in FLAC3D use a Coulomb sliding criterion where the cohesion and friction angles are constant (Itasca, 2019). The shear and normal stiffnesses were set to 1×10^{12} (one tenth of the equivalent stiffness of the stiffest neighbouring zone) (Roberts, 2022). Figure 4.8 shows the positions of the three interfaces.

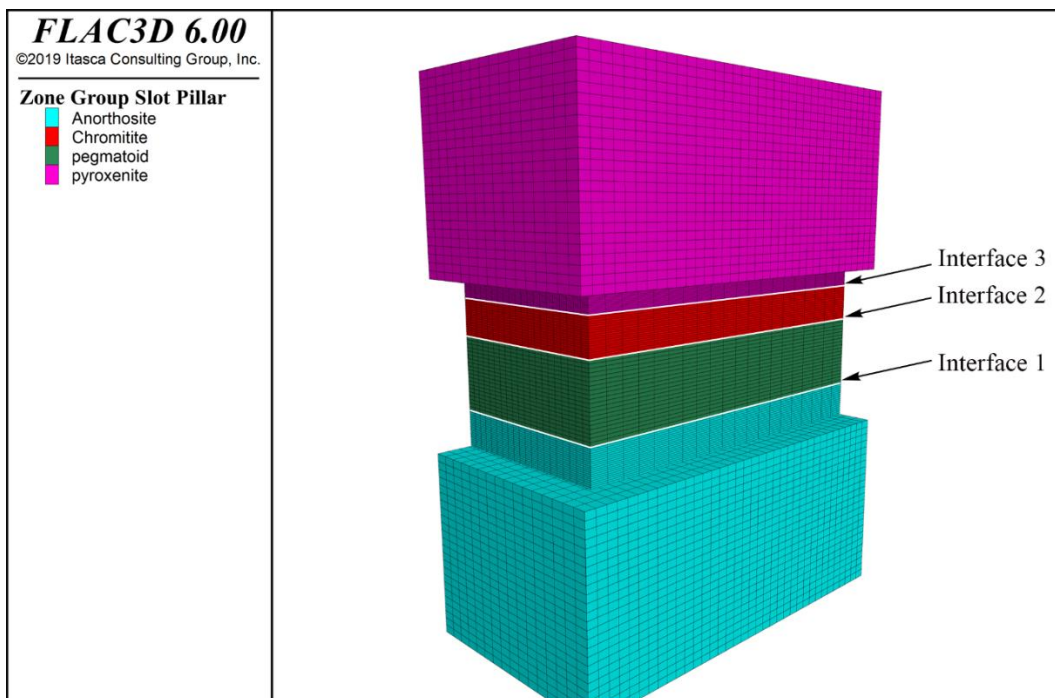


Figure 4.8 Schematic showing the three interfaces

For the initial model runs, the interfaces were assumed to have the same cohesion and friction as the weaker lithology. These are presented in Table 4.7 and the results are shown in Figure 4.9.

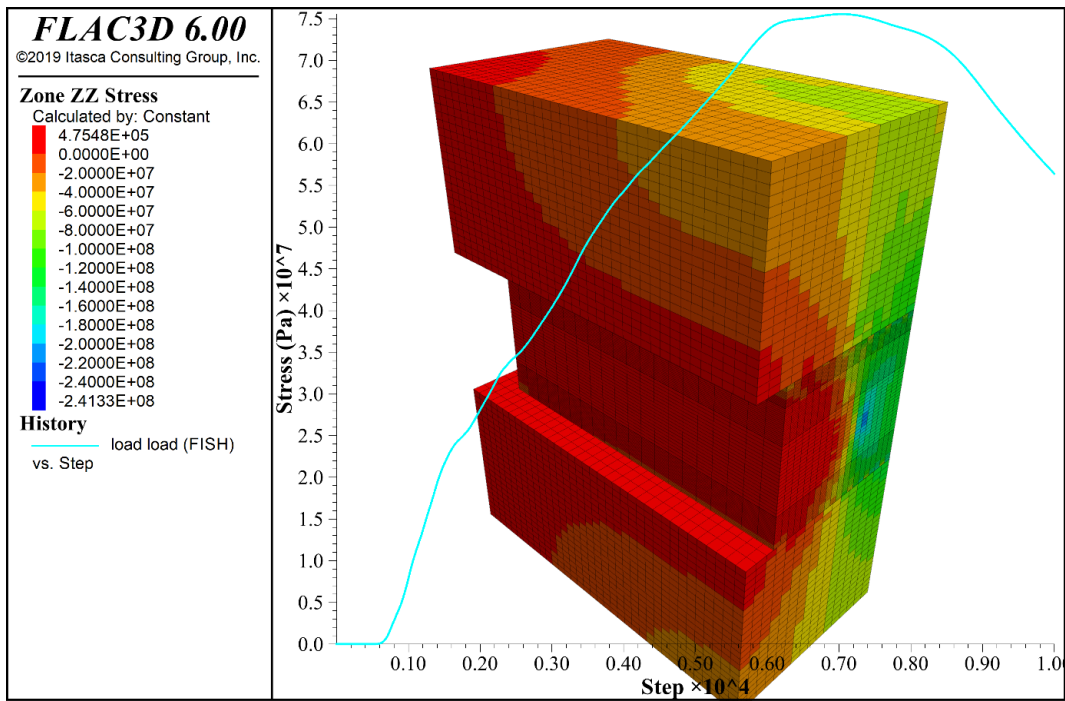


Figure 4.9 Initial model results

Table 4.7 Interface input parameters

Interface	Cohesion	Internal friction angle
Anorthosite-pegmatoid (1)	4.9	32.8
Pegmatoid-chromitite (2)	4.9	32.8
Chromitite-pyroxenite (3)	10.5	42.39

The model underestimates the peak strength as some of the factors that influence strength are not accounted for. The interfaces influence how the lithologies interact. The failure of the pillar initiates from the weakest lithology (pegmatoid) starting from the edge of the pillar progressing towards the core. This is observed in Figure 4.10. Figure 4.10 shows the plastic shear strain accumulation. The interfaces were set to values in Table 4.7.

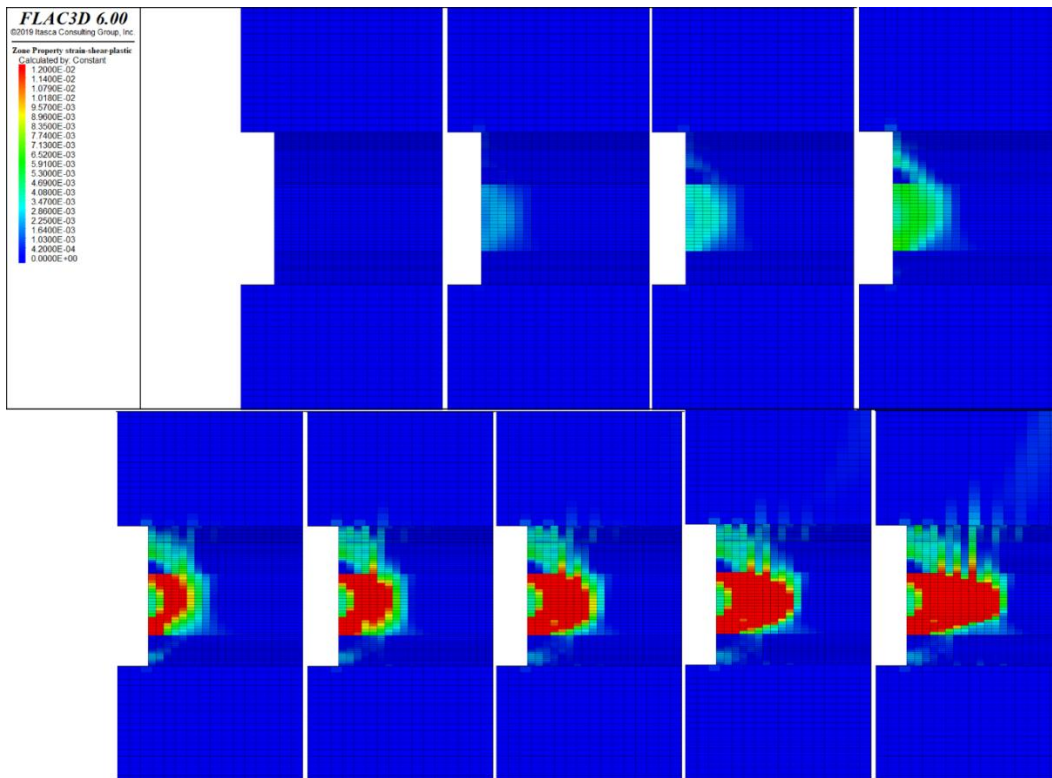


Figure 4.10 Progression of failure

Pillar failure initiates from the edge of the pillar in the pegmatoid layer, progressing into the chromitite layer and is concentrated in these two weaker layers. The failure continues to progress through the pillar towards the core as the pillar exceeds peak strength. At this stage, the failure seems to progress into the hangingwall (pyroxenite) suggesting possible hangingwall damage. The vertical stress follows the same trend as failure, initially

concentrated at the edge and subsequently progressing towards the core. This is shown in Figure 4.11.

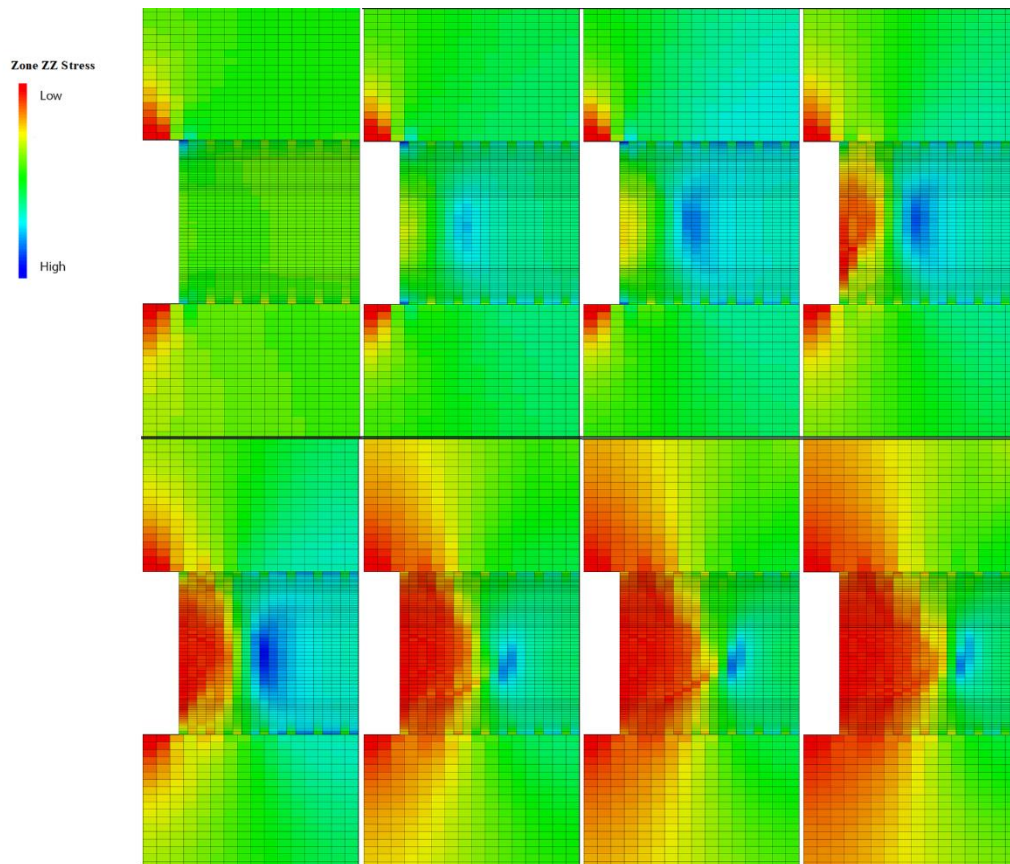


Figure 4.11 Stress variation as pillar fails

The progression of failure between the different rock layers appears to be controlled by the interfaces. A similar effect was also observed with stress. To investigate the influence of the interface, the input parameters were set to cohesion of 1 MPa and a friction angle of 30° for all three interfaces. The model was then run at friction angles of 20° and 10° for each interface whilst the other interfaces were kept at 30°. Reducing the friction angle makes the interface more slippery and this should have some influence on the pillar. The same was repeated with the cohesion at 5 MPa and 10 MPa. The friction angles were

kept constant at 30° . Running the model with all three interfaces set to the cohesion of 1 MPa and friction angle of 30° provided similar results, as shown in Figure 4.9, meaning that the interfaces have little effect if the friction angle is 30° .

Multiple runs were done on all three interfaces varying the cohesion between 5 MPa and 10 MPa. The results suggest that cohesion has little effect on the peak strength and the progression of failure through the pillar. The results obtained from varying the cohesion with the friction angle set to 30° were the same as those shown in Figure 4.9. Additional model runs were made for cohesions of 20 MPa and 30 MPa, and no significant changes were observed. However, varying the friction angle below 30° has significant influence on both the peak strength and the progression of failure.

4.3.2.1 Interface 1 (Anorthosite-Pegmatoid)

Interface 1 (Figure 4.8) has the most influence on peak strength. Reducing the friction angle to 20° resulted in a slight drop in strength. Further reducing the friction angle to 10° resulted in a more significant drop in strength, dropping by 5 MPa. Varying the friction angle affects the post peak part of the curves. This is shown in Figure 4.12.

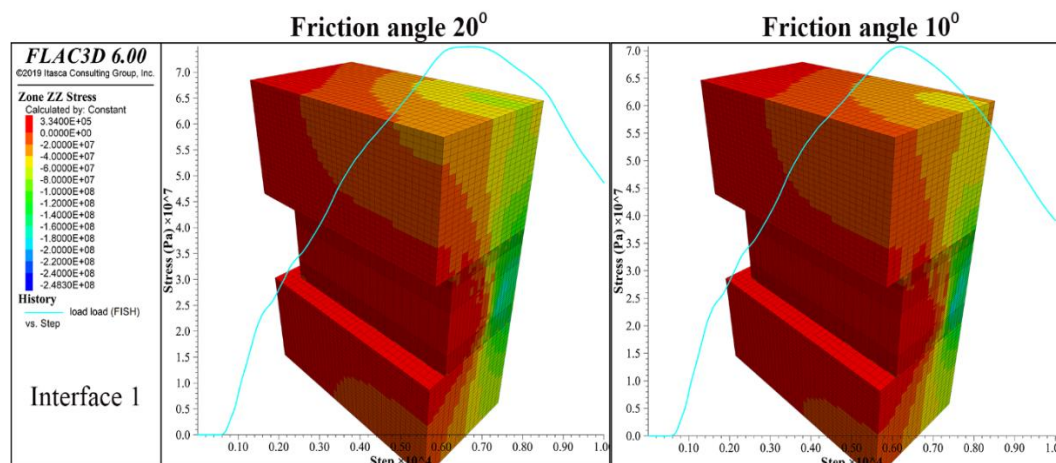


Figure 4.12 Influence of interface 1 on peak strength

The interface controls the amount stress and damage transferred to the other zones and lowering the friction angle lowers the stress transferred and increases the rate of damage progression. The extent of damage was greatest when the friction angle was 10° . This is shown in Figure 4.13. This suggests that the interface has influence on the strength of the core of the pillar. If the friction angle on an interface is low, this may result in a weaker core.

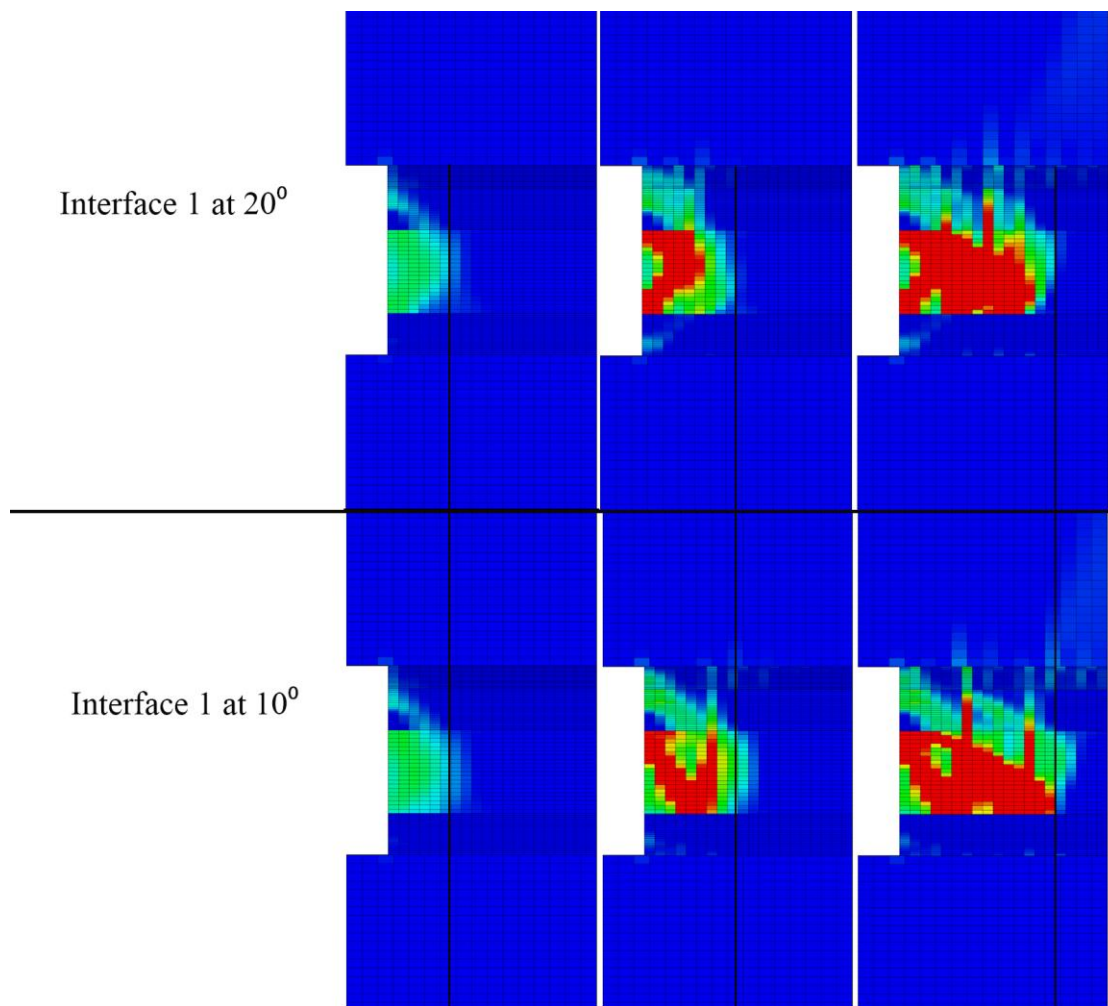


Figure 4.13 Influence of interface 1 on progression of failure

4.3.2.2 Interface 2 (Pegmatoid-Chromitite)

Lowering the friction angle to 20° on the second interface (Figure 4.8) has little effect on the peak strength and on the post-peak part of the curve. Models run with the interface set to a friction angle of 10° show some effect on the peak strength and post peak characteristics. The peak strength dropped by 2 MPa. Figure 4.14 shows the model results for interface 2.

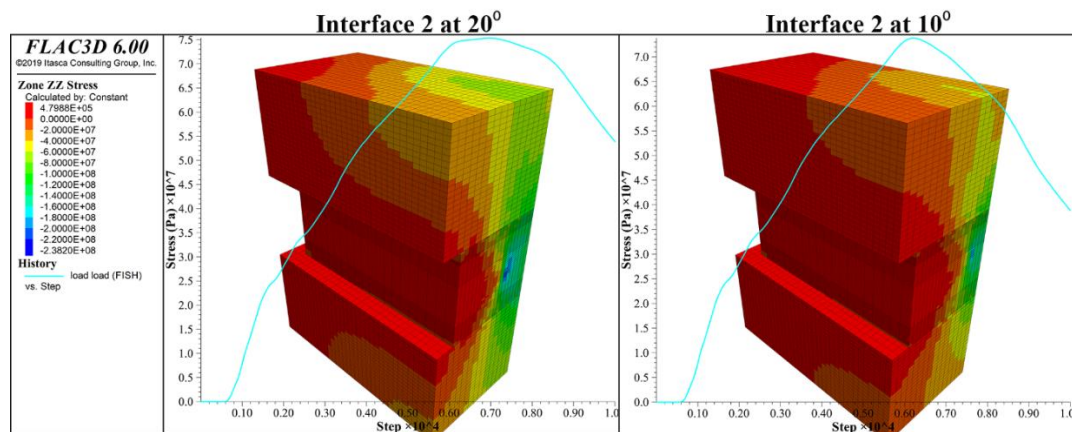


Figure 4.14 Influence of interface 2 on peak strength

Lowering the friction angle on interface 2 restricts damage to the weaker rock layer. The extent of this effect increases as the friction angle decreases. The extent of damage is greater when the friction angle is low and restricted in the pegmatoid layer. This may explain the small effect interface 2 has on peak strength. This is illustrated in Figure 4.15. Interface 2 has significant influence on the damage that progresses into the hangingwall.

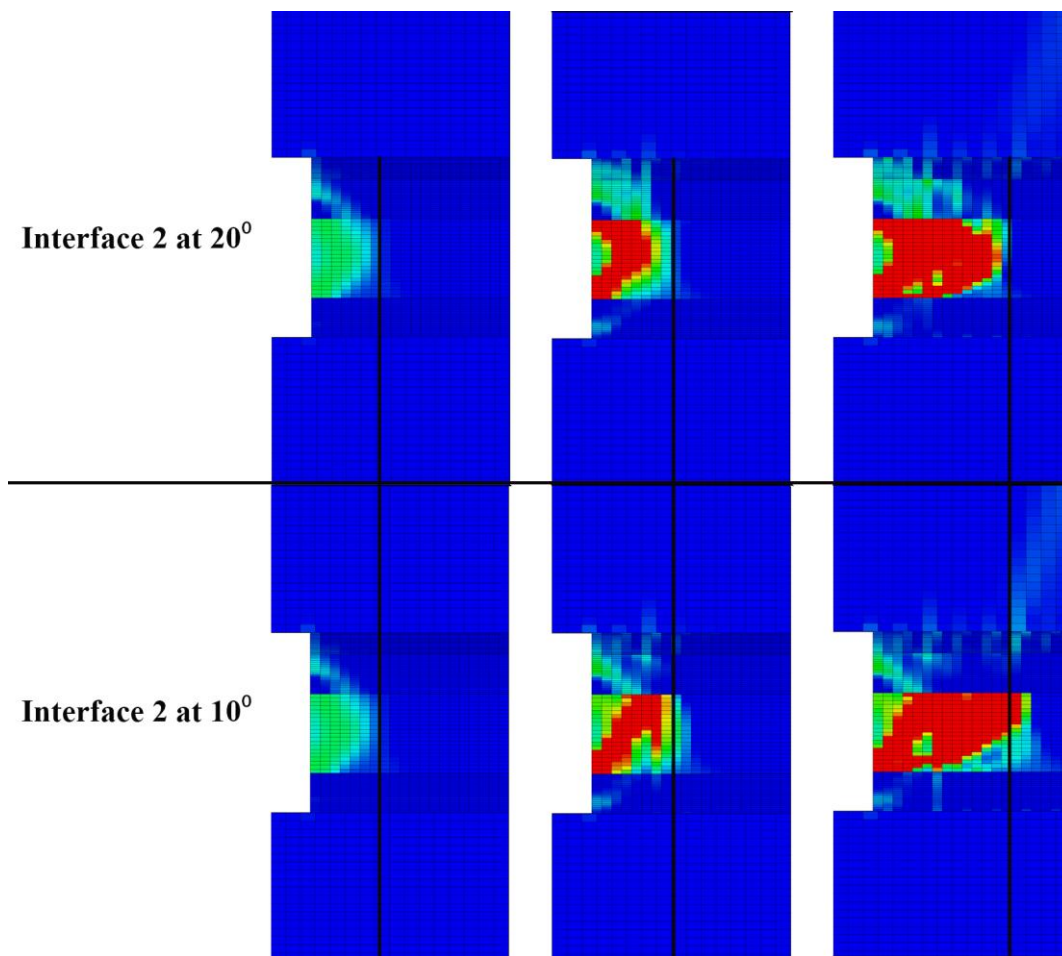


Figure 4.15 Influence of interface 2 on progression of failure

4.3.2.3 Interface 3 (Chromitite-Pyroxenite)

Setting the friction angle to 20° on the third interface (Figure 4.8) has the same effect as the first interface on peak strength (1 MPa drop). Lowering the friction angle to 10° results in a small decrease of strength (drops by 3 MPa). The third interface has limited effect on the post-peak behaviour. Figure 4.16 shows the influence of interface 3 on peak strength at friction angles of 20° and 10°.

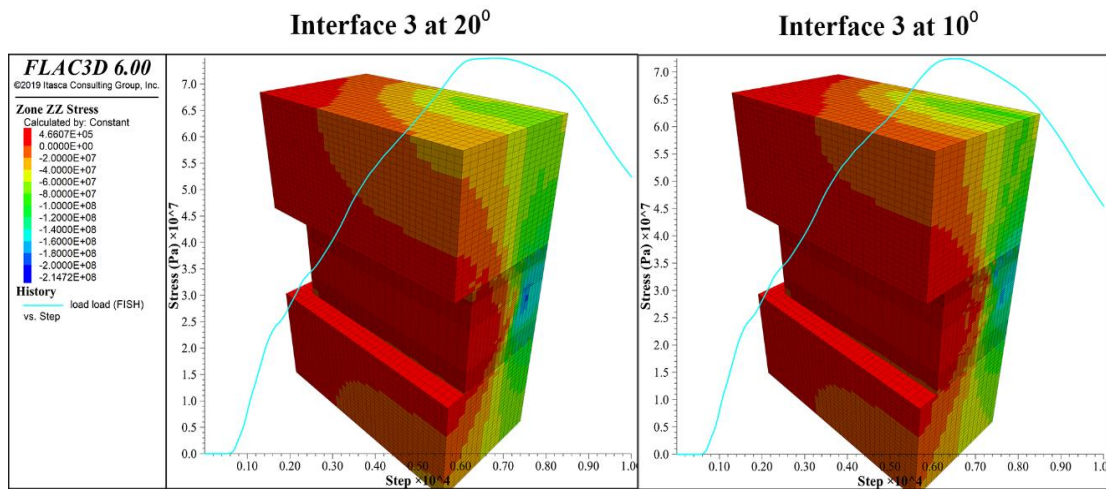


Figure 4.16 Influence of interface 3 on peak strength

Lowering the friction angle on interface 3 concentrates the damage in the two weaker layers (chromitite and pegmatoid). At friction angle of 10° the hangingwall damage is greater than at 20°. This is illustrated in Figure 4.17. The extent of pillar damage is to some extent controlled by the friction angle of the interface. All three interfaces have little effect on the footwall. The position of the interface dictates the extent to which the interface can influence the pillar.

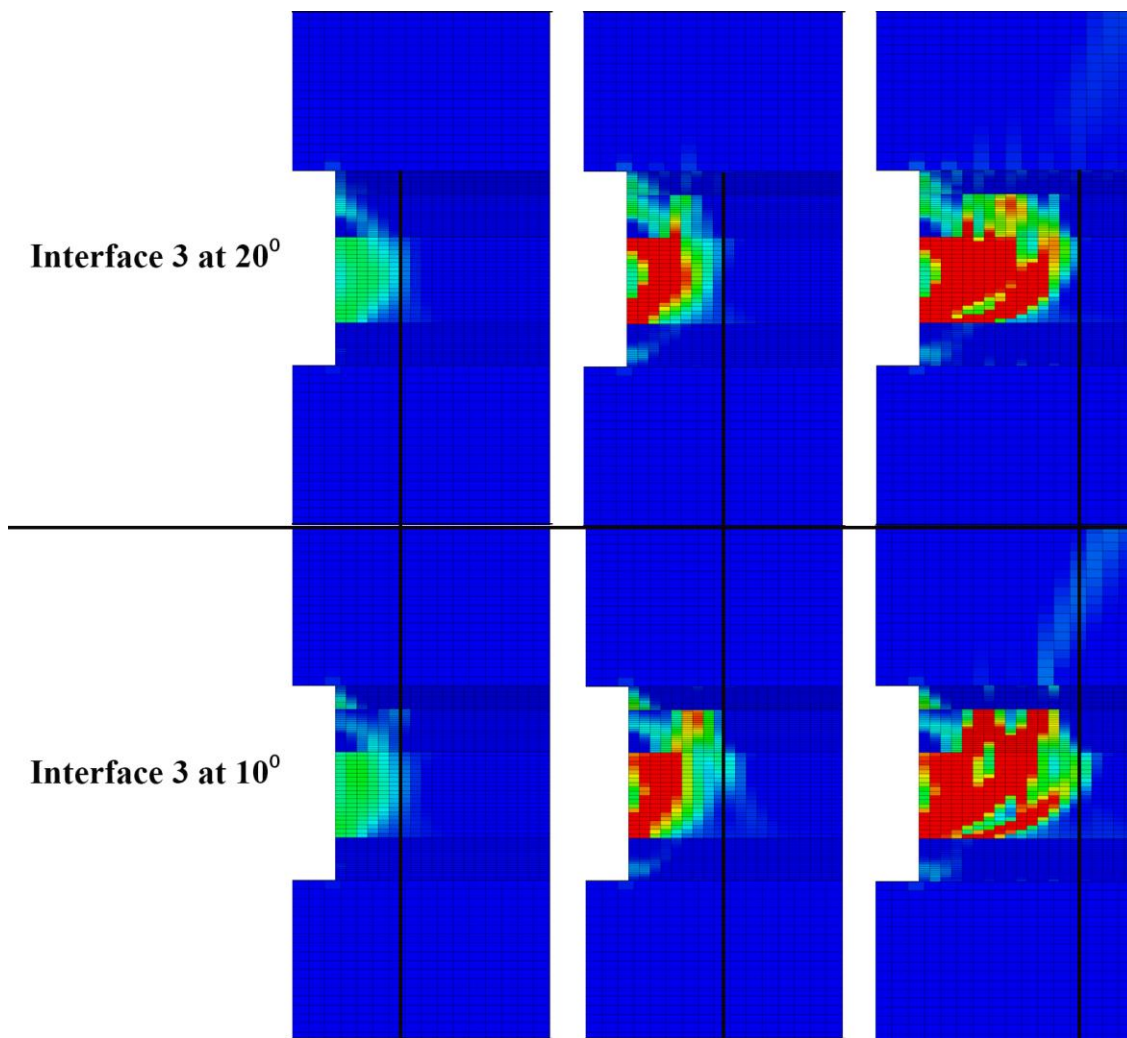


Figure 4.17 Influence of interface 3 on progression of failure

The evidence from the model suggests that interfaces have significant influence on the behaviour of a pillar and should be considered when modelling pillar behaviour. Results of tilt tests conducted in Chapter 3 are reproduced in Table 4.8 and were used as the friction angles for the interfaces. The cohesion of the interface has little effect on the behaviour of the pillar for friction angles of 30 degrees or greater and was set to 1 MPa for all three interfaces.

Table 4.8 Interface input values

Interface	Cohesion (MPa)	Internal friction angle (°)
Anorthosite-pegmatoid (1)	1	30
Pegmatoid-chromitite (2)	1	33
Chromitite-pyroxenite (3)	1	35

4.3.3 Model description

The model was setup in FLAC3D using primitive grid generation. Ratios were used in the grid generation to concentrate fine mesh around the pillar. The mesh size increases as the distance from the pillar increases. This was done to prevent the model from having an excessive number of zones. Uniform fine mesh was used in the pillar. The dimensions of the pillar are 4 m x 2 m x 0.91 m. The full span of the stope is included in the model. The length and width of the foundations (hangingwall and footwall) are 30 m x 5.5 m. The hangingwall has a thickness of 5 m whilst the footwall is 4.8 m. 1.5 m holing (0.75 m on either side of the length of the pillar) was included. Gully dimensions of 1.3 m wide and 1.8 m deep were used (Carollo, 2022). The constitutive model was set to the Mohr-Coulomb (strain softening) criterion. The strain softening model is based on the cohesion-weakening, friction-strengthening model (Hajiabdolmajid et al., 2002). Roller boundaries were applied on the sides and base of the model. A velocity of 10^{-6} m/s was applied at the top of the models. Figure 4.18 shows the plan view of the modelled area. The shaded

rectangles show the region modelled for each scenario. Two scenarios were modelled, Scenario 1 has a 2 m siding whilst Scenario 2 has no siding (Figure 4.18). To simplify the model, zero dip is assumed in both scenarios.

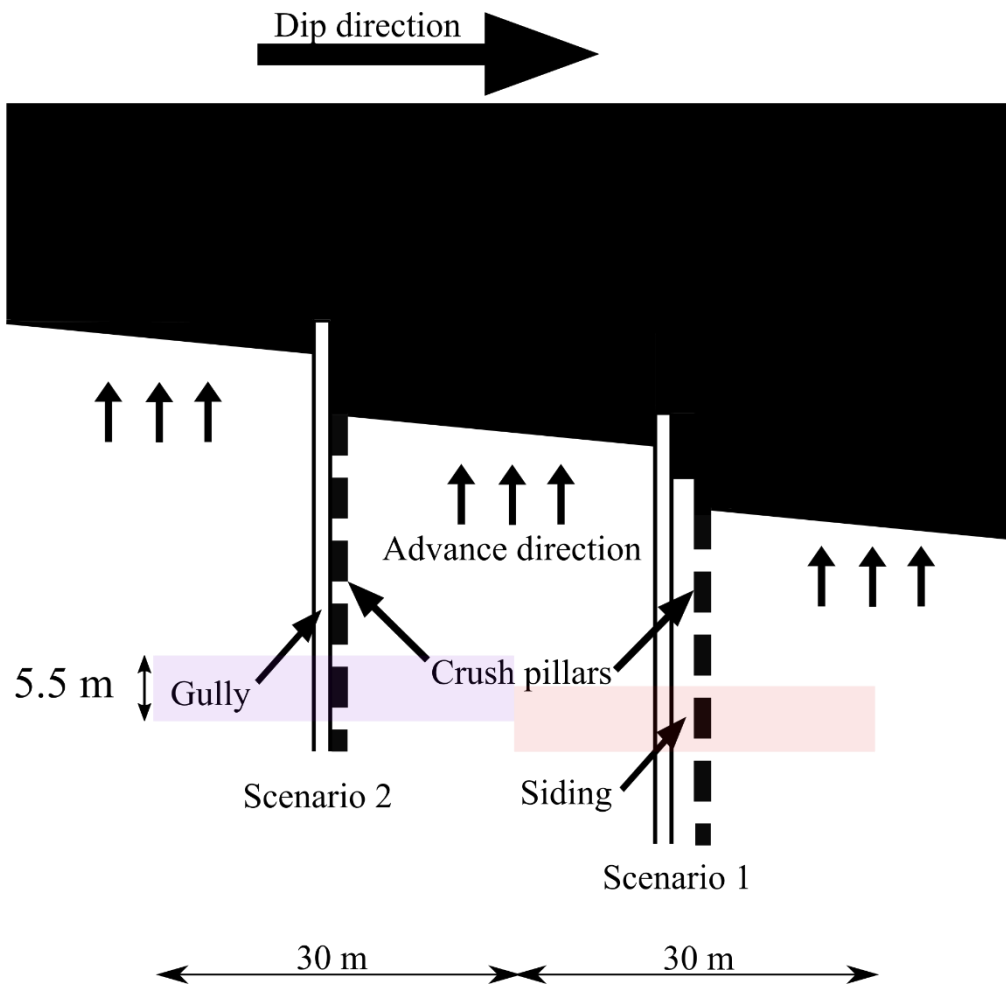


Figure 4.18 Schematic (plan view) showing the two modelled scenarios

The residual cohesion and friction angle values used in the model were determined using Mohr's circle (Figure 4.2). For simplicity, the dilation and friction angle were kept constant at residual values. Figure 4.19 shows the variation of cohesion, dilation angle and friction angle with plastic strain. The solid lines represent the friction angle, the dashed lines

represent cohesion and the dotted lines represent the dilation angle. Each rock type is assigned a colour:

- Blue represents chromitite;
- green represents anorthosite;
- red represents pyroxenite; and
- black represents pegmatoid.

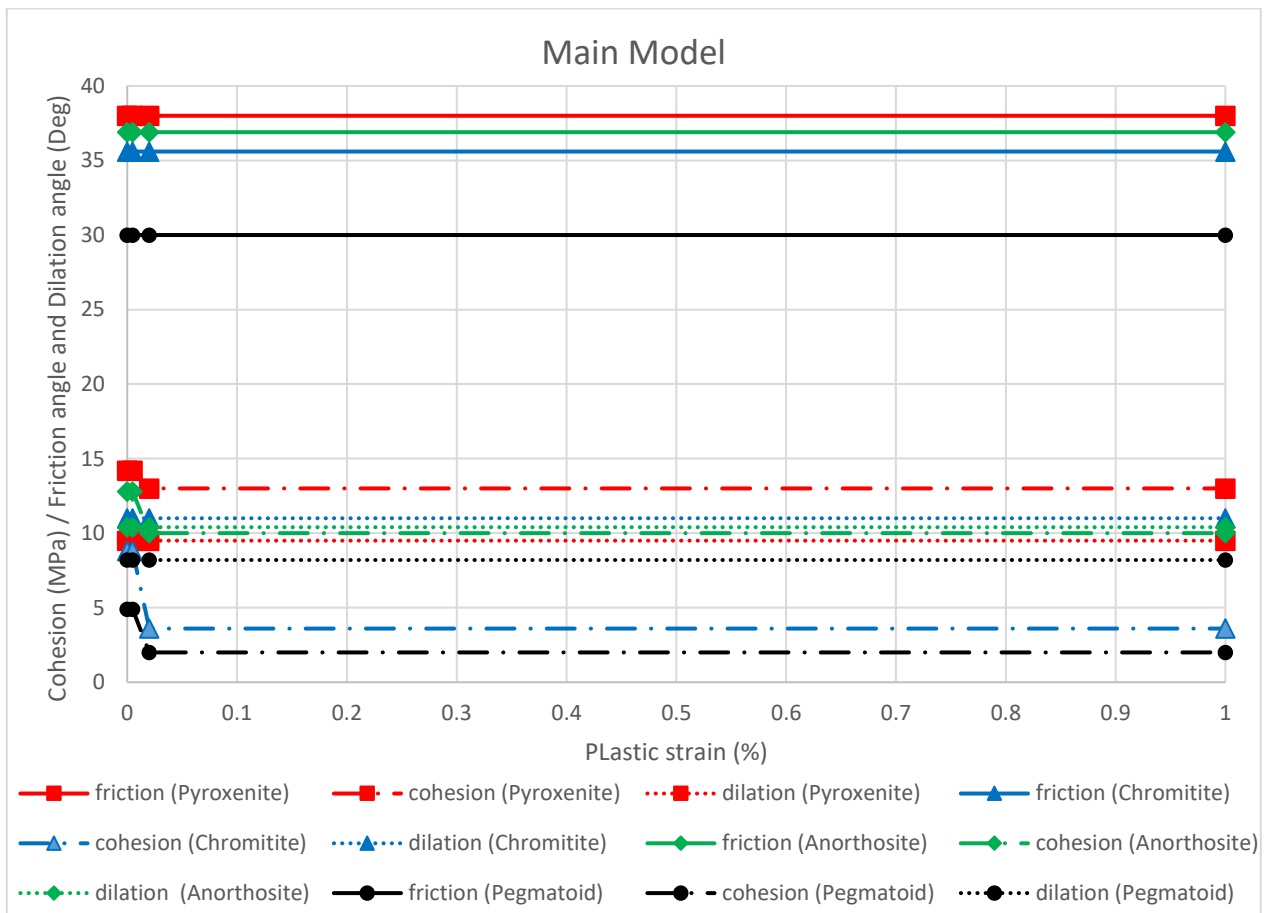


Figure 4.19 Shows the strain softening curves

The strain softening curves in Figure 4.19 resulted in the model (pillar) exhibiting perfectly plastic behaviour as shown in Figure 4.20. This was not in line with crush pillar behaviour and necessitated the adjustment of the residual cohesion. This was achieved by making

the cohesion approach zero as the plastic strain approaches 1%. This is illustrated in Figure 4.21. The adjustments were based on pillar modelling example provided in the FLAC3D manual (Itasca, 2019). The strain softening curves in Figure 4.21 were used in the main model.

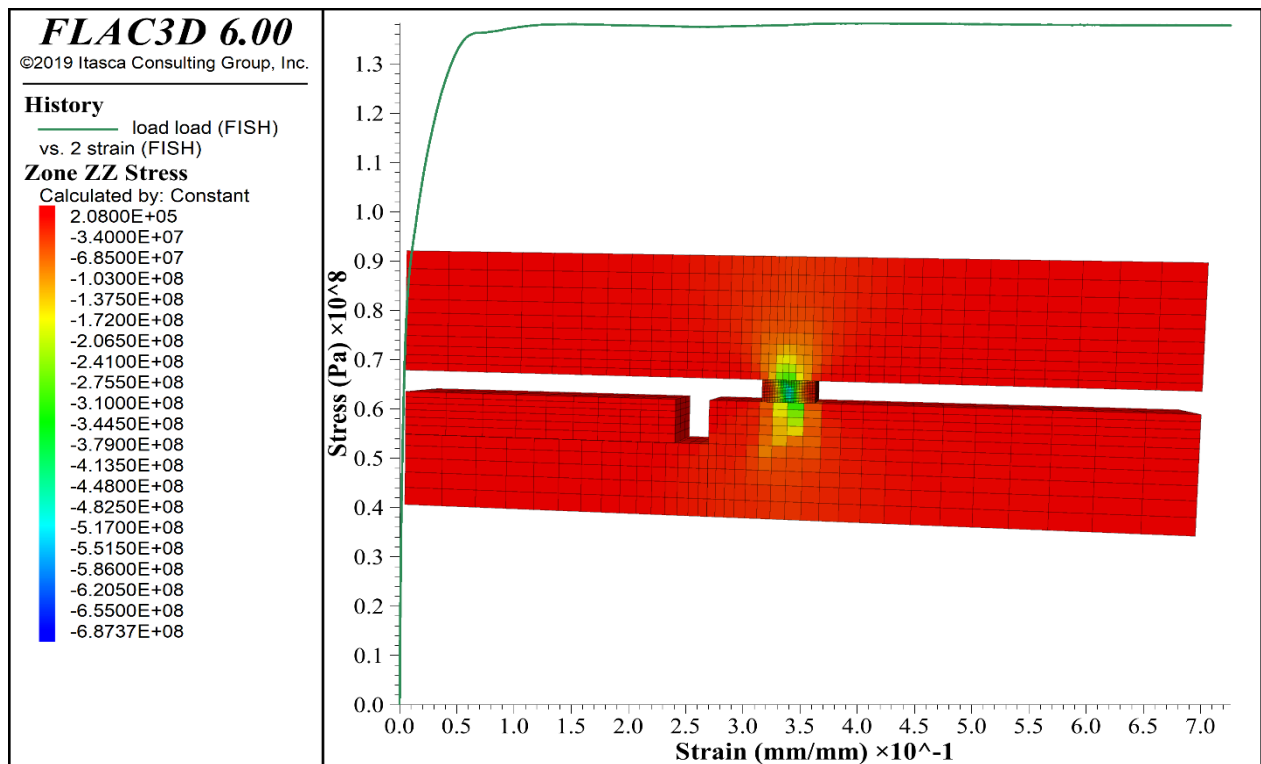


Figure 4.20 Model showing perfectly plastic behaviour.

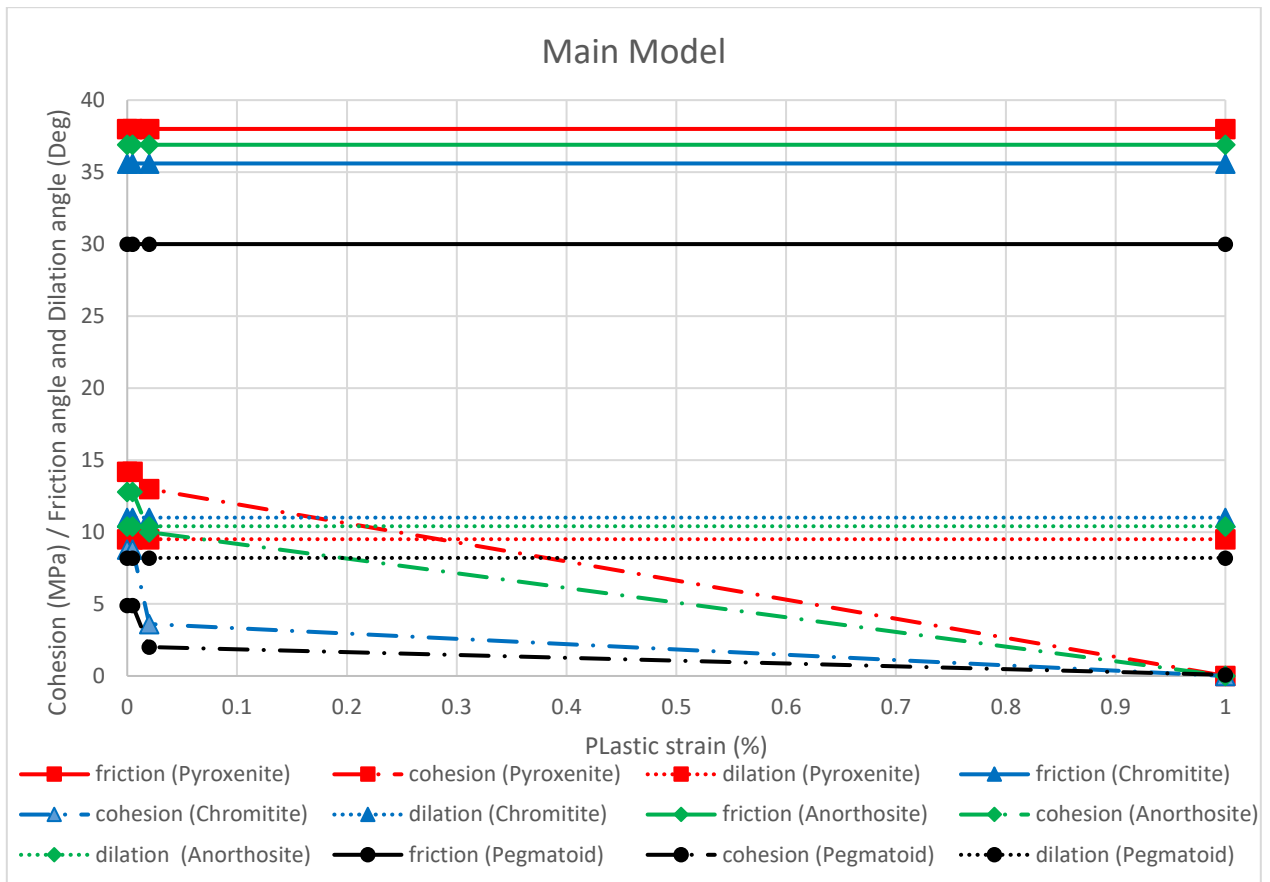


Figure 4.21 Strain softening curves used in modelling pillar behaviour

4.3.4 Scenario 1

Scenario 1 models a case where a 2 m siding is present between the gully and the pillar. Sidings are either used to change the fracture patterns resulting from high face stress or to protect people from falling rock by having some distance (1 m - 2.5 m) between the crush pillars and the travelling way (Du Plessis and Malan, 2018). The configuration of Scenario 1 is shown in Figure 4.22.

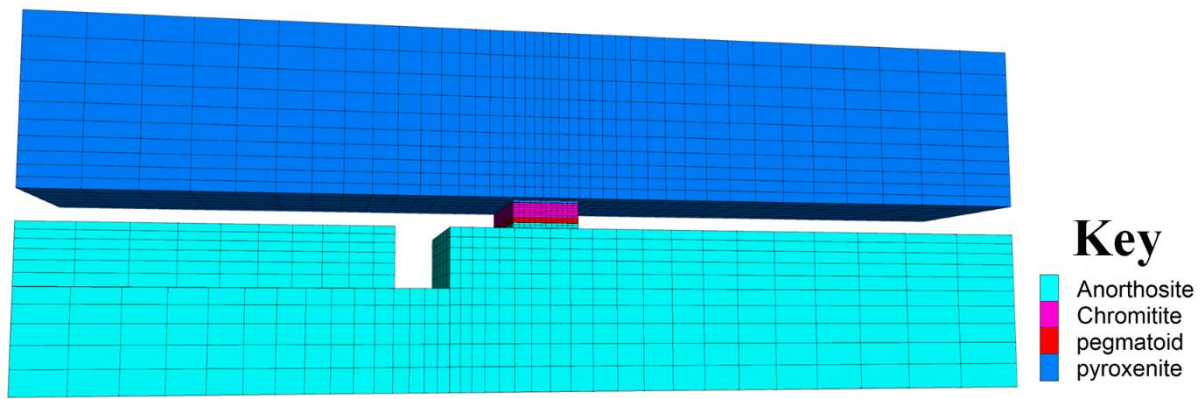


Figure 4.22 3D Model of Scenario 1

The result of the model estimates a peak strength of 104 MPa and a residual strength of about 12 MPa. Figure 4.23 shows the model results for Scenario 1. Pillar failure initiates from edges of the pillar towards the core. This may be exaggerated by the bending of strata around the pillar causing the so-called draping effect mentioned by Watson (2010) (Figure 4.24). This results in high initial stresses in the edges of the pillar. As the pillar approaches peak strength, the stress concentrates in the core of the pillar. This could also be exaggerated by the extra confinement provided by the draping effect (Figure 4.25). At this stage significant damage is observed in the foundations. This could be caused by the pillar trying to expand as it experiences further compression. This results in inducing tensile stresses in the foundations creating tensile fractures (Hertzian cracks) (Watson, 2010). The pillar enters the post-peak phase when peak strength is exceeded and it starts to shed its load. In post-peak phase the stress concentration in the pillar reduces as the pillar approaches residual strength (Figure 4.25). The extent of damage on the foundations is also observed to increase, reaching the base of the gully, as the pillar approaches residual strength (Figure 4.24). At residual strength, no significant damage

progression is observed. The stress concentration in the pillar is at its lowest since the peak strength of the pillar has been exceeded. Figure 4.24 shows the failure progression through the pillar. Figure 4.25 shows the variation of stress as the pillar fails.

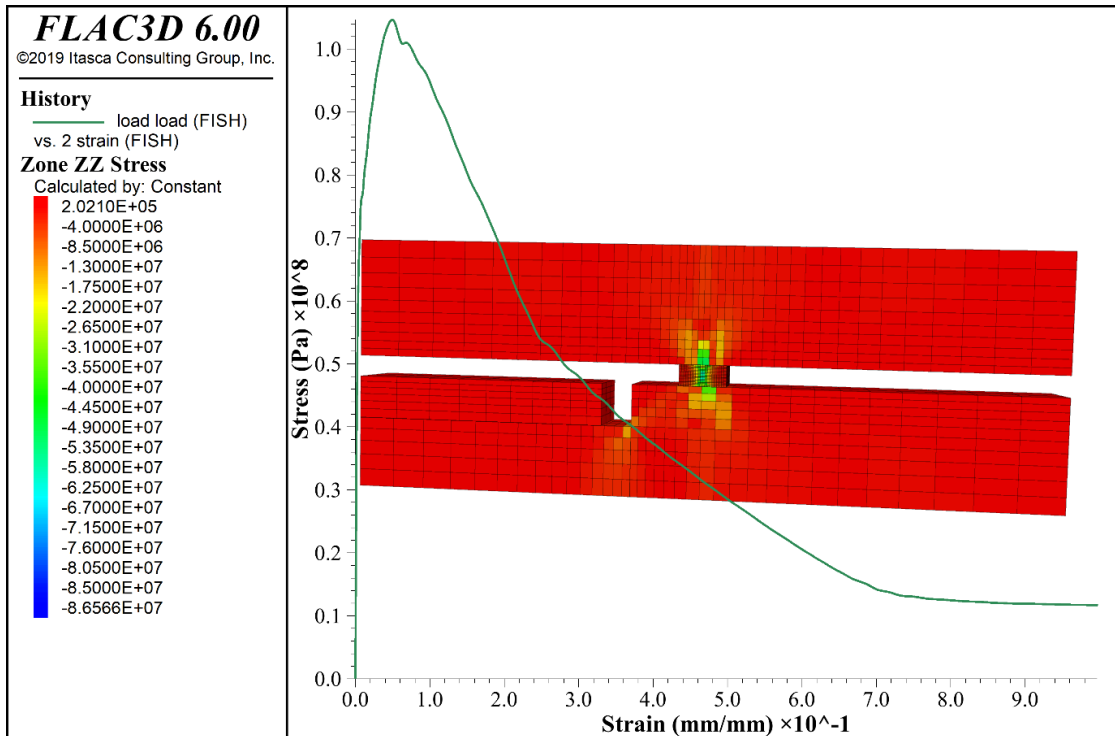


Figure 4.23 Model results for Scenario 1

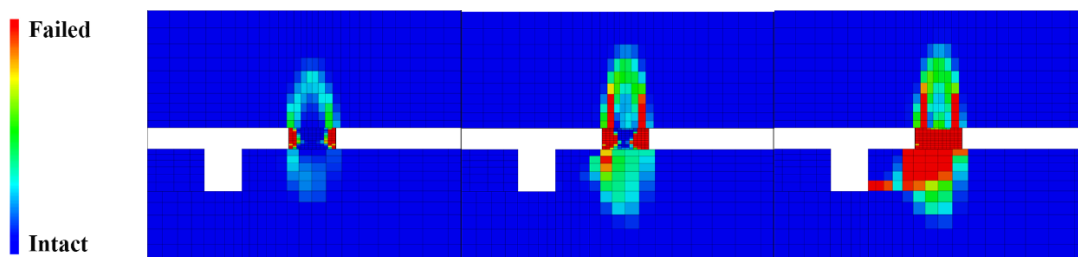


Figure 4.24 Progression of failure Scenario 1

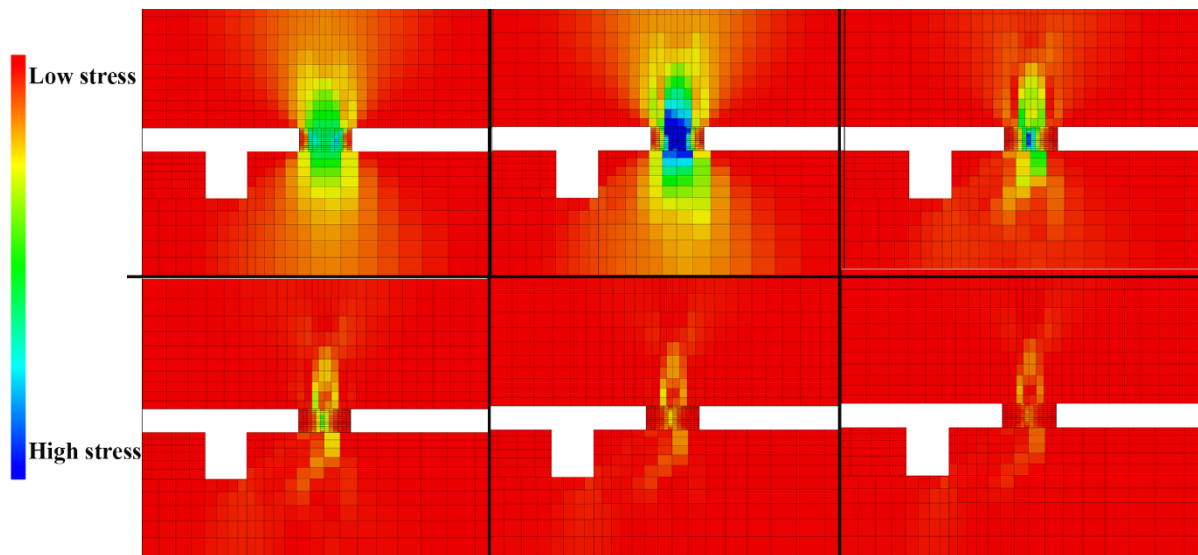


Figure 4.25 Variation of stress Scenario 1

4.3.5 Scenario 2

Sidings are difficult to clean and the process involves hand lashing. For this reason, it would be easier and quicker to mine crush pillars without a siding (Du Plessis and Malan, 2018). Scenario 2 models a case where no siding is present next to the pillar. Figure 4.26 shows the configuration of scenario 2.

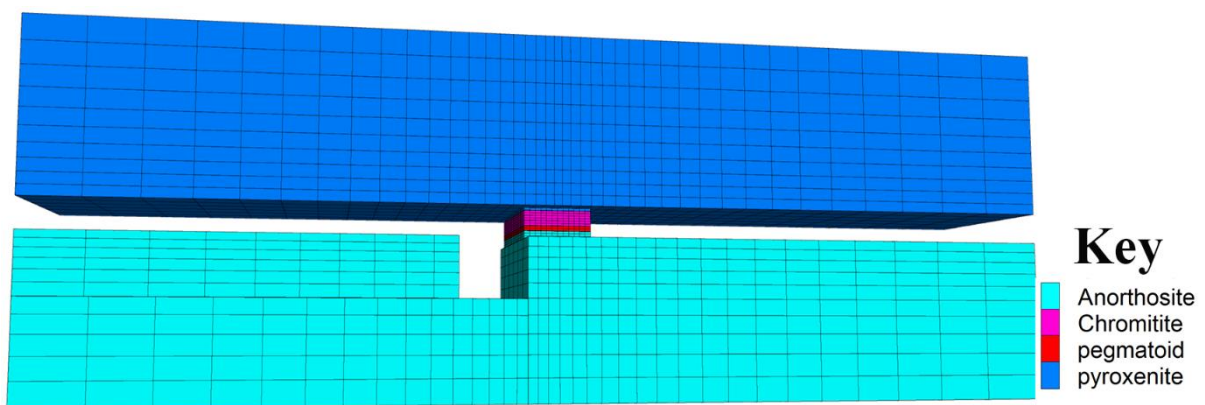


Figure 4.26 Model setup for Scenario 2

The model results for Scenario 2 estimate a peak strength of 79 MPa and a residual strength of 12 MPa. Underground stress measurement may be required to check if the residual strength is sufficient to arrest back breaks. The peak strength of Scenario 2 is significantly lower than that of Scenario 1, suggesting a significant effect of a siding. The adjacent gully changes the height of the pillar on one side and increases the effective height of the pillar. This in turn reduces the w/h ratio making the pillar weaker. The model results for Scenario 2 are shown in Figure 4.27.

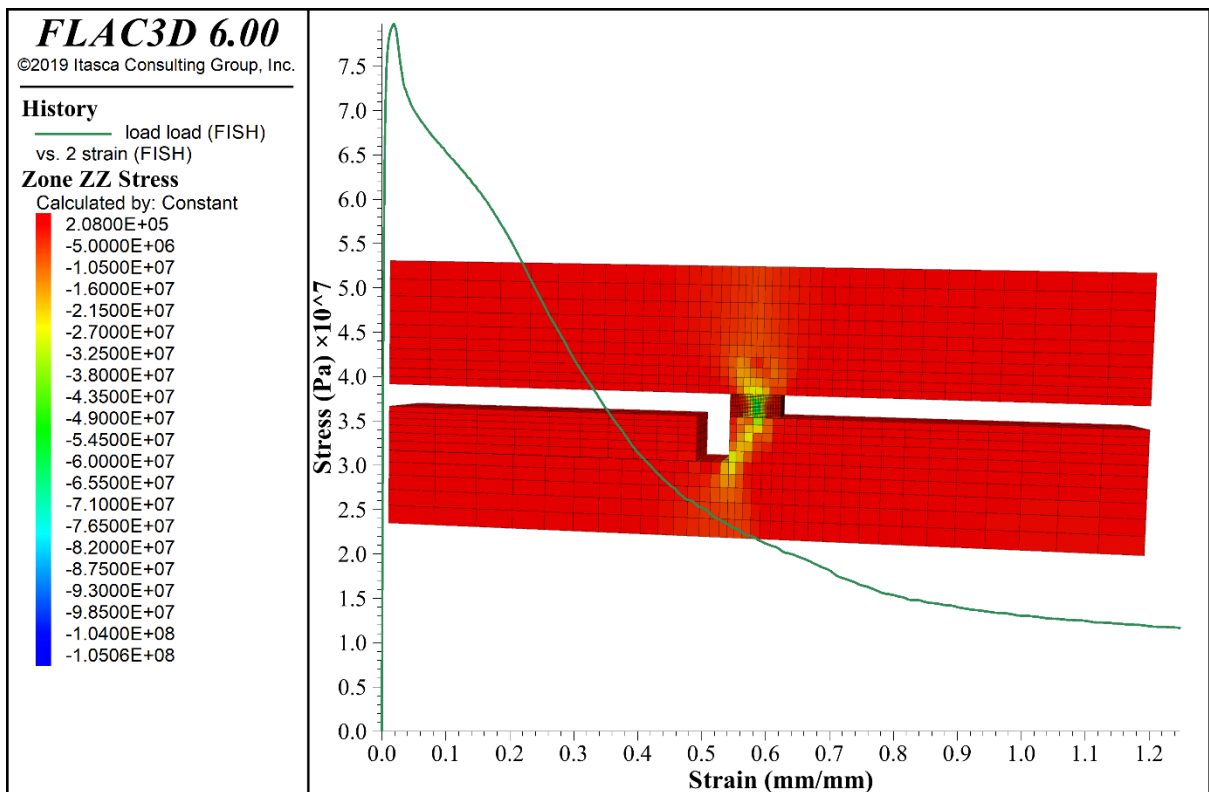


Figure 4.27 Model results for Scenario 2

Evidence of draping can be seen in the early stages of pillar deformation. The edges of the pillar are under high stress whilst the core is at low levels of stress as shown in Figure 4.28. In Scenario 2 the progression of failure is concentrated in the footwall. This is

probably caused by the lack of confinement due the presence of the gully. Hangingwall damage is observed but not to the extent of the footwall. The failure progression pattern is similar to Scenario 1 in the early stages of deformation. The pillar failure initiates from the edges of the pillar progressing towards the core. As the stress levels approach peak, more deformation is observed in the footwall progressing into the gully (Figure 4.29). In the post-peak region, very little change in deformation is observed. The stress is high in the core of the pillar as the load approaches peak strength. In the post-peak region, the stress level drops gradually until it reaches residual strength (Figure 4.30). The model suggests possible issues of footwall heave maybe encountered. The model also suggests that the pillar would fail into the gully.

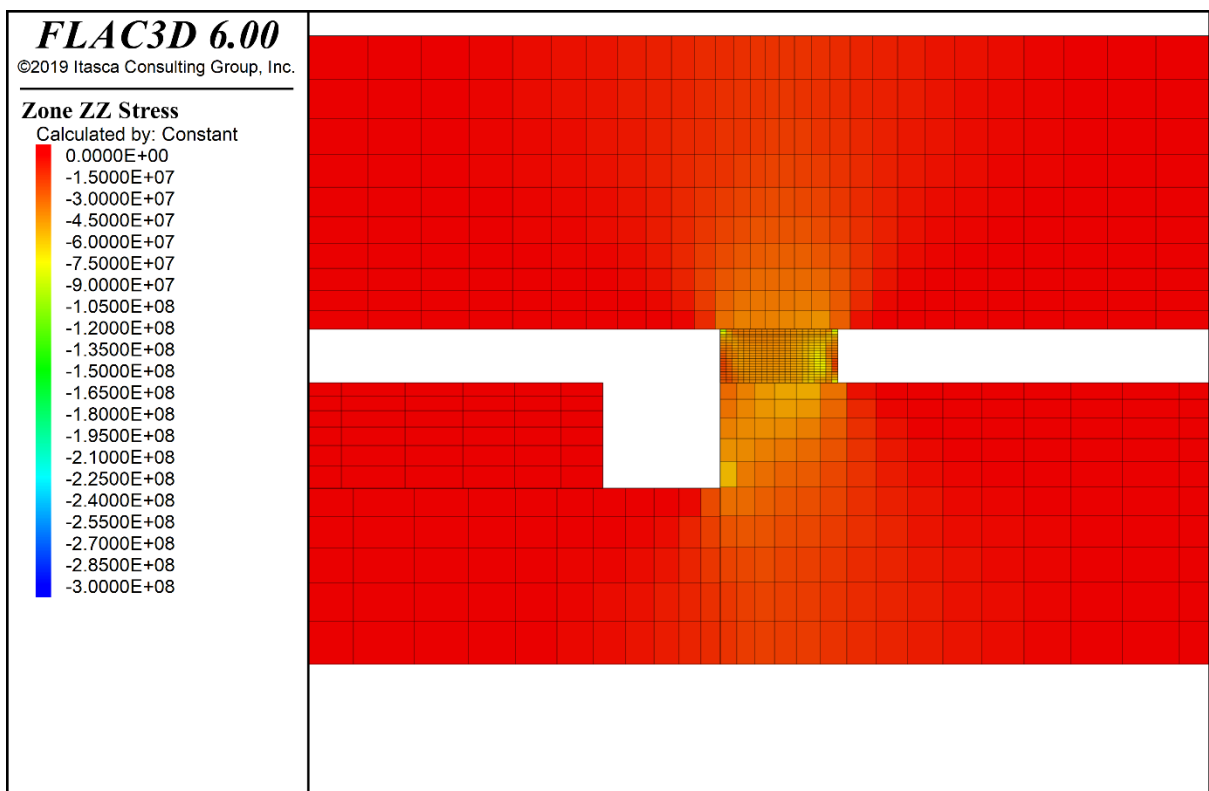


Figure 4.28 Draping effect

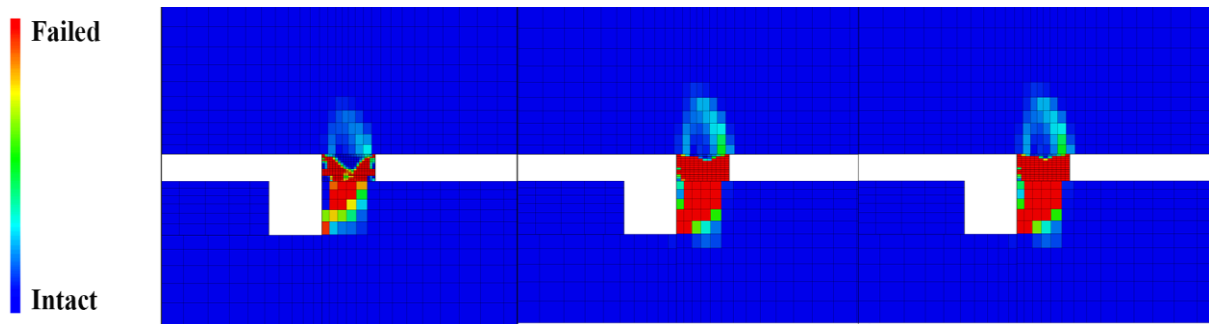


Figure 4.29 Progression of failure Scenario 2

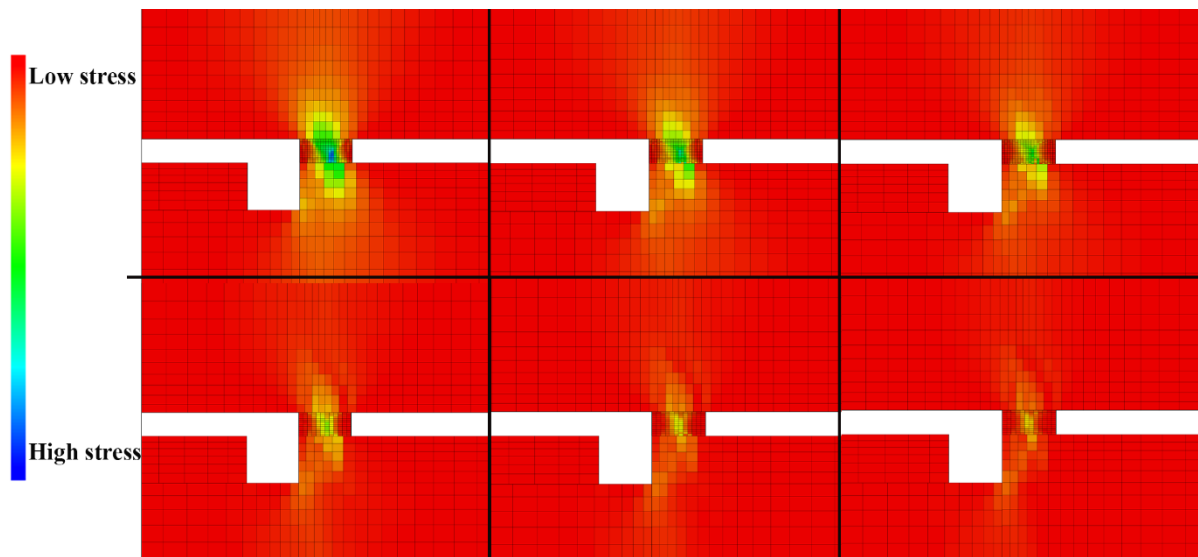


Figure 4.30 Stress variation for Scenario 2

The formula suggested by Watson et al. (2021) was used to estimate the peak strength of the pillar (section 2.8).

$$Pillar\ strength = Kw_e^\alpha h_e^\beta \quad (4-7)$$

Where $h_e \approx \left[1 + 0.2692\left(\frac{w}{h}\right)^{0.08}\right]$

$$w_e \approx \frac{2wL}{(w + L)}$$

K = in situ cube strength

w = pillar width

h = pillar height

L = pillar length

$\alpha = 0.67$

$\beta = -0.32$

The formula estimated peak strengths of 132 MPa for Scenario 1 and 107 MPa for Scenario 2. The predicted strength by the platmine formula is higher than predicted by the model (in both scenarios), and further work needs to be done to understand why there is such a difference. The difference between the platmine formula and the model for a pillar with and without a siding is 28 MPa.

4.3.6 Safe depth to introduce crush pillars

The first step in understanding whether the stresses are high enough to cause pillar failure at the face was to determine the span across the centre-raise where the first in-panel pillars would be cut. After completion of the centre-raise, a ledging cycle cuts a 0.9 m high ledge on either side, which extends the span (Figure 4.31). The first crush pillars would be cut only after the ledging cycle is complete. As these pillars are being cut, fracturing

and controlled yielding must initiate, and the pillar must reach residual strength before the loading conditions become too soft. The safe depth to introduce crush pillars refers to the depth where stresses are high enough (exceed peak strength of pillar) to initiate controlled failure of the first crush pillars near the face. The ledging standard at Impala Mine stipulates that the ledge should be 10 m wide. Figure 4.31 shows a cross-section view of the centre gully and ledges on either side.

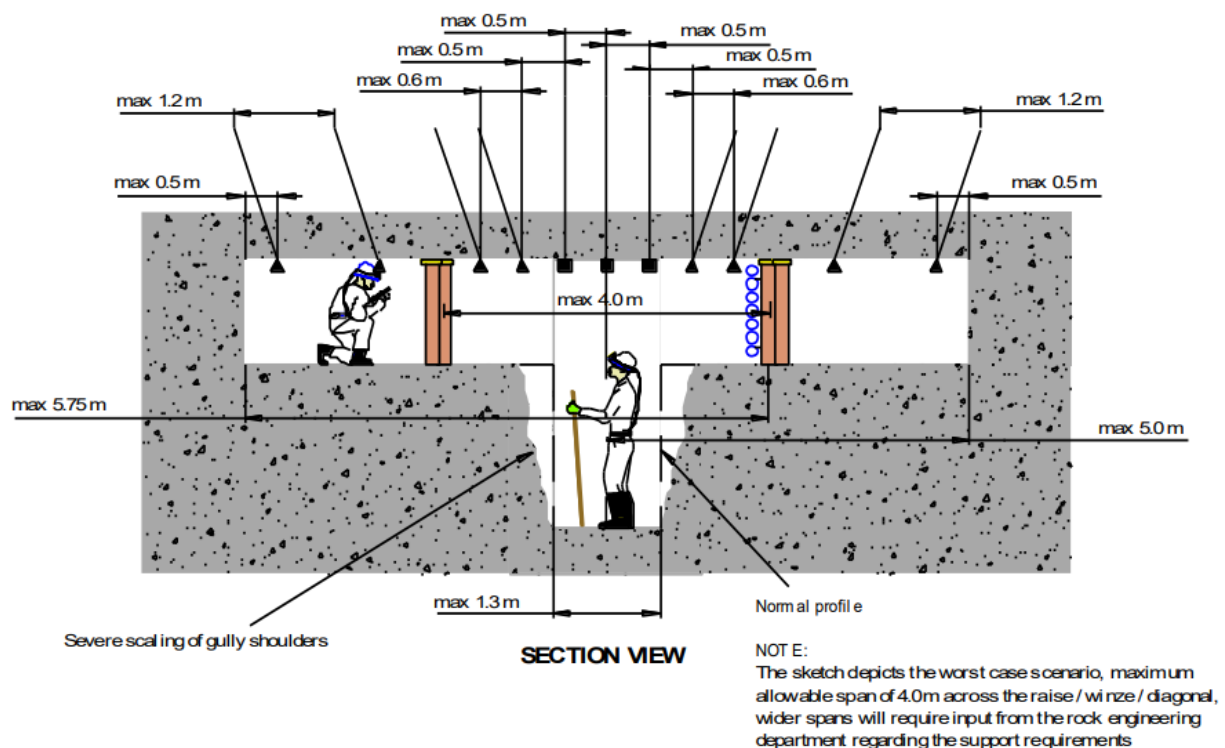


Figure 4.31 Schematic of the ledging process at Impala Mine (Carollo, 2022)

Two scenarios were considered to determine the depth to safely introduce crush pillars after ledging: the worst-case and the best-case scenarios. The worst-case scenario refers to the stress conditions present around the first line of crush pillars after ledging of the raise line surrounded by the large volume of unmined ground. The size of the unmined

ground is such that the stress level returns to virgin further away from the raise line. This scenario would have the least available stress. The best-case scenario refers to the stress conditions present around the first line of crush pillars after ledging of a raise line through an unmined stope surrounded by mined out ground. This scenario would have the most available stress. The two scenarios should bracket the actual stress conditions. The models are based on work done by Watson et al., 2010 that involved stress investigations on 4 m – 5 m stubs. During the study, two stubs were observed to fail.

Worst-case scenario

A model was set up in FLAC3D to estimate the minimum depth crush pillars can be introduced soon after ledging. The model dimensions were 214 m x 32 m x 40.91 m. Along the width, two half pillars (4 m x 1 m) were located at the edges 30 m apart. This was done to insure repeating geometries. Along the length, at the centre is a 10 m ledge with a 4 m x 1 m stub next to it (Figure 4.33). On either side of the ledge and stub is 100 m of unmined ground. The dimensions of the model were chosen to ensure that the stress levels at the edge of the model (along length) would be virgin. Fine mesh was used on the pillars and the size of the mesh was made to increase further from the centre. Roller boundaries were applied around the model except at the top where a velocity of 10^{-6} m/s was applied. The constitutive model was set to elastic. Figures 4.32 – 4.33 show the different views of the model. Stress measurements were taken at the centre of the pillar (highlighted in green as shown in Figure 4.33) and across the top of the model (highlighted in red as shown in Figure 4.32). The stress measurement across the model was used to estimate the virgin stress. The model was run until the stress experienced by the pillar exceeded its peak strength.

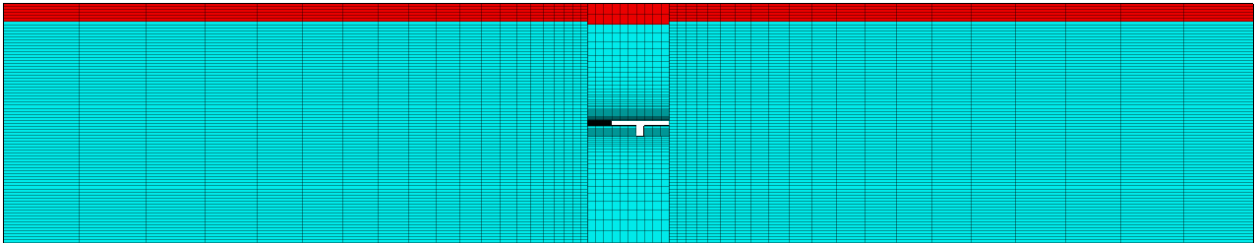


Figure 4.32 Full model side view

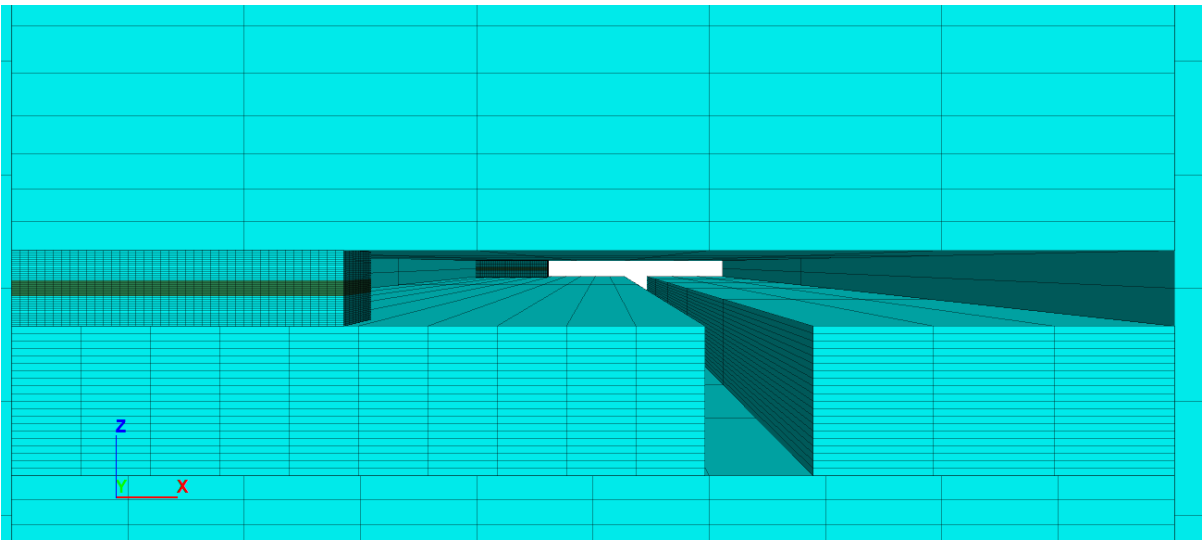


Figure 4.33 Close up view of the ledge and the two stubs

The results of the worst-case scenario model are shown in Figure 4.34. The red curve represents the stress measured across the entire model and was used to estimate the virgin stress. The green curve represents the stress measured at the centre of the pillar. The dashed line shows the virgin stress that corresponds to 104 MPa (pillar with siding), and the solid line shows the virgin stress that corresponds to 79 MPa (pillar with siding). To estimate the virgin stress where the pillar would fail, Figure 4.34 was used to estimate the virgin stresses that correspond with the peak stresses of 104 MPa (with siding) and

79 MPa (without siding). This is illustrated in Figure 4.34. The model is representative of a pillar without a siding; however, the absence of a siding should be insignificant since the model is elastic. The model estimated the pillar would fail at virgin stresses of 19.5 MPa and 14 MPa for models with and without a siding, respectively. To determine the depth that corresponds with the estimated virgin stress, equation 4-8 was used.

$$\text{Virgin stress} = \text{gravity} \times \text{density} \times \text{depth} \quad (4-8)$$

Where gravity = 9.81 m/s²

Average rock density= 3000 kg/m³

Equation 4-8 was re-arranged, making depth the subject of the formula, and was used to estimate the depth. The results estimated that crush pillars can be safely introduced after ledging at depths of 663 m and 476 m for models with and without a siding, respectively.

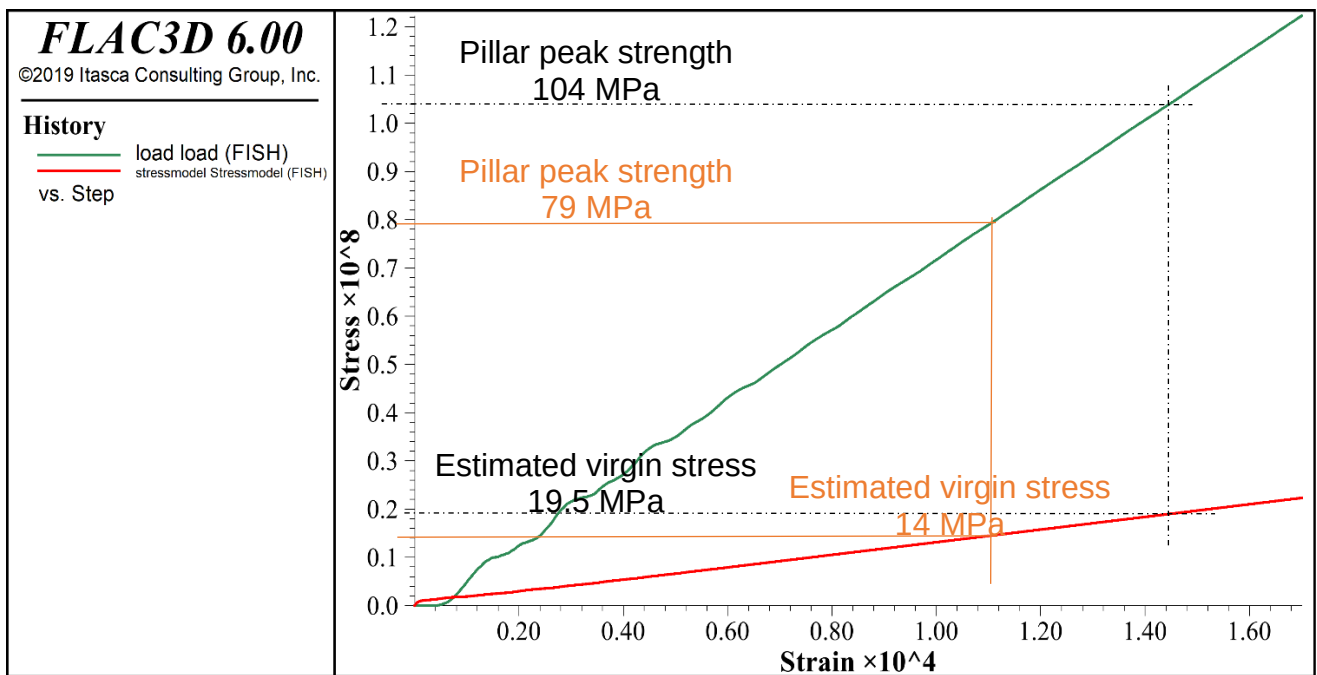


Figure 4.34 Worst-case scenario model results

Best-case scenario

A model was set up in FLAC3D to estimate the minimum depth crush pillars can be introduced. The model dimensions were 596 m x 32 m x 40.91 m. The model is similar to the worst-case scenario model, with the only difference being 150 m of unmined ground with 150 m mined out stope on either side. This is shown in Figure 4.35 and 4.36. The boundaries and stress measurements were applied the same as in the previous model. The model was run until the stress experienced by the pillar exceeded its peak strength.

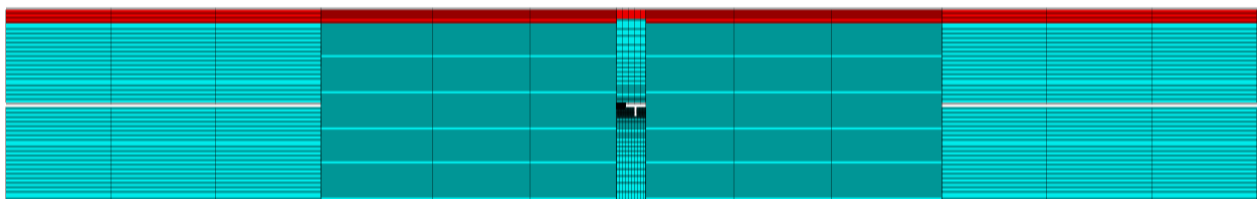


Figure 4.35 Side view of the best-case model

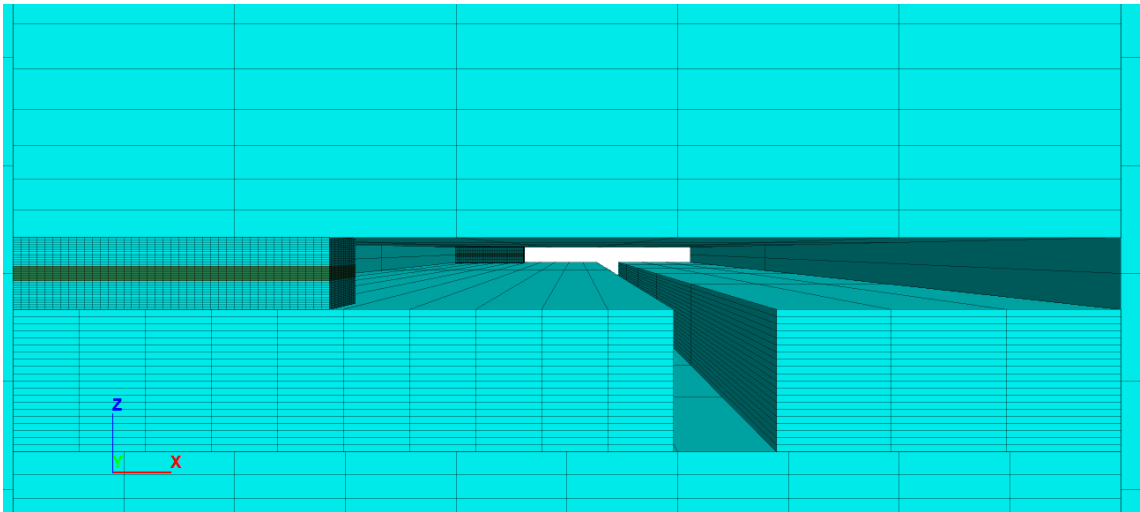


Figure 4.36 Close up view of model

The best-case scenario model results are shown in Figure 4.37. Like in the previous model, the red and green curves represent the stress measured across the model and the stress measured at the centre of the pillar, respectively. Figure 4.37 estimated the virgin stresses corresponding with the peak stresses of 104 MPa (represented by a dashed line) and 79 MPa (represented by a solid line) for models with and without a siding, respectively. This is illustrated in Figure 4.37. The model estimated the pillar would fail at virgin stresses of 10.5 MPa and 8 MPa for models with and without a siding, respectively. Equation 4-8 was used to determine the depths from the virgin stresses and were found to be 357 m (pillar with siding) and 272 m (pillar without siding). This means the crush pillars can be used from depths of 357 m (pillar with siding) and 272 m (pillar without siding). However, at shallow depths (< 663 m for pillars with a siding and 476 m for pillars without a siding), great caution must be exercised when introducing crush pillars at these depths because they fall within the transition zone. In the transition zone, the available stress may not be sufficient, posing the risk of pillar bursting.

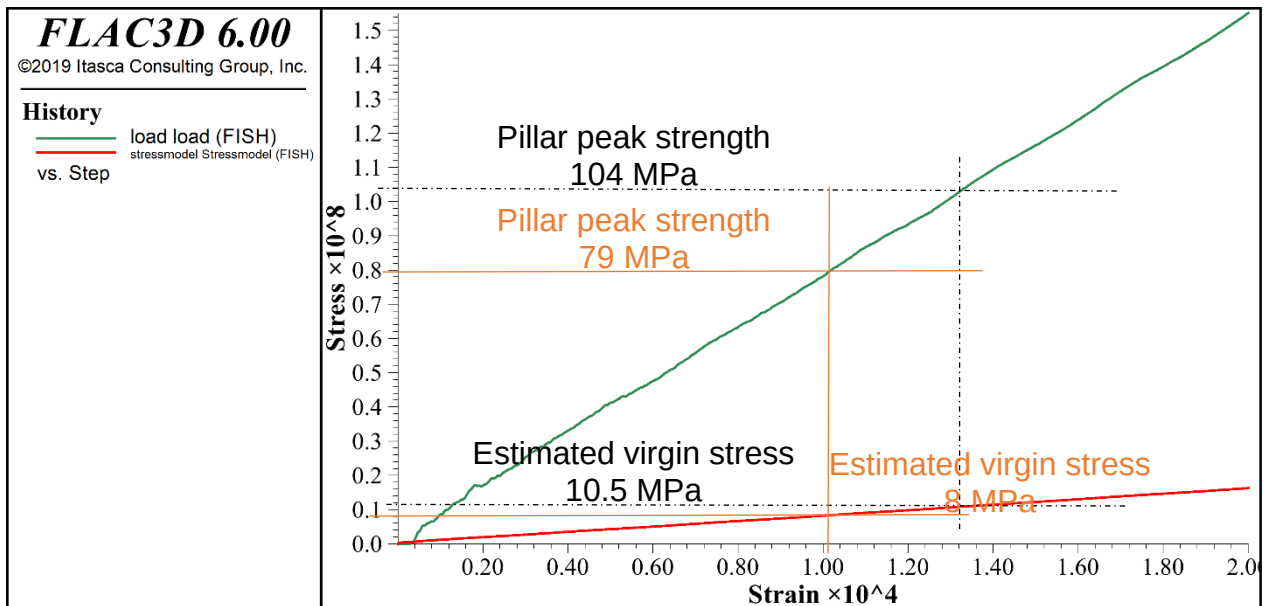


Figure 4.37 Best-case scenario model results

4.4 Chapter summary

Six sets of models were formulated in FLAC3D. The first model was setup to reproduce stress-strain curves obtained from the laboratory triaxial test, using input parameters from Chapter 3. The second model analysed the effects of interfaces between different rock types in the pillar. Sensitivity analyses suggested that the friction angle of the interface has a greater influence than the cohesion on the progression of failure, and hence peak strength of the pillar. The third and fourth models simulated the two 2 m x 4 m crush pillar scenarios, i.e., with and without a siding. These models were used to analyse the strength of, and failure progression through, a pillar. The models estimated peak strengths of 104 MPa and 79 MPa for pillars with and without a siding, respectively. The residual strength of the pillars was estimated to be about 12 MPa. The fifth and sixth models (Worst-case and best-case scenarios) were used to estimate the depths crush pillars can be used. The worst-case scenario model estimates that the crush pillars can be safely introduced from depths of 663 m for pillars with a siding and 476 m for pillars without a

siding. The best-case scenario model estimates that the crush pillars can be used at shallow depths from 357 m (pillar with a siding) and 272 m (pillar without a siding), but caution must be exercised. The stresses in the transition zone may not be sufficient to initiate failure and the available stress is dependent on the extent of the mining in the surrounding workings.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

The research reported in this thesis contributes to the future of South African platinum mines that use crush pillars. The major finding of the research was the strength of 2 m x 4 m multi layered UG2 pillars in a stoping height of about 1 m. The aim of the investigation was to determine at what depth below surface such pillars could be introduced. The analysis was done using rock cores from the mine, which were tested in the Geomechanical Test Laboratory of the University of the Witwatersrand. The geomechanical properties of the rock types in the pillar as well as the immediate hangingwall and footwall were used as input parameters to FLAC3D models. These models not only provided the strength of the pillars, but also made it possible to track the failure progression through the pillar and foundations as the applied stress was increased. The main findings of the research showed:

- The dilation angle tends to decrease with increasing confinement, but no effect was observed on the Poisson's ratio;
- Pillar failure initiates from the edge of the pillar and progresses towards the core;
- The interfaces between different lithologies influence how different layers interact. However, the cohesions of these interfaces appear to have little to no effect on the pillar behaviour;
- The friction angle on the planes has a significant influence on pillar behaviour and peak strength. Varying this friction influences the progression of failure;

- The change in internal friction with plastic strain (post failure) had little effect on pillar strength, but the post peak behaviour was heavily controlled by cohesion loss.
- Two scenarios for the crush pillar were modelled. Scenario 1 assumes the presence of a 2 m wide siding. Scenario 2 assumes the pillar is right next to the gully. For Scenario 1 and 2 the estimated peak strength was found to be 104 MPa and 79 MPa, respectively;
- Crush pillars can be safely introduced at depths of 663 m and 476 m for pillars with and without sidings, respectively.
- The pillars can be used at shallower depths (from 357 m for pillar with a siding and 272 m for the one without a siding) but caution must be exercised.

5.2 Recommendations

The following recommendations are made for future research:

- It is recommended that a series of suitable laboratory tests be conducted on the interfaces that exist between the lithologies for UG2.
- Additional triaxial tests with post-peak at higher confinements for the rock types on the UG2 are recommended. This would add more post-peak parameters to the available database.
- A series of suitable underground tests and stress measurements are recommended to confirm the peak and residual values on pillars.
- This study did not investigate what stiff conditions are required for a crush pillar and how these conditions vary with distance from the face. It is recommended that future studies investigate this.

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APPENDIX A

Appendix A1- Stress-strain graphs for pyroxenite, pegmatoid, chromitite and anorthosite.

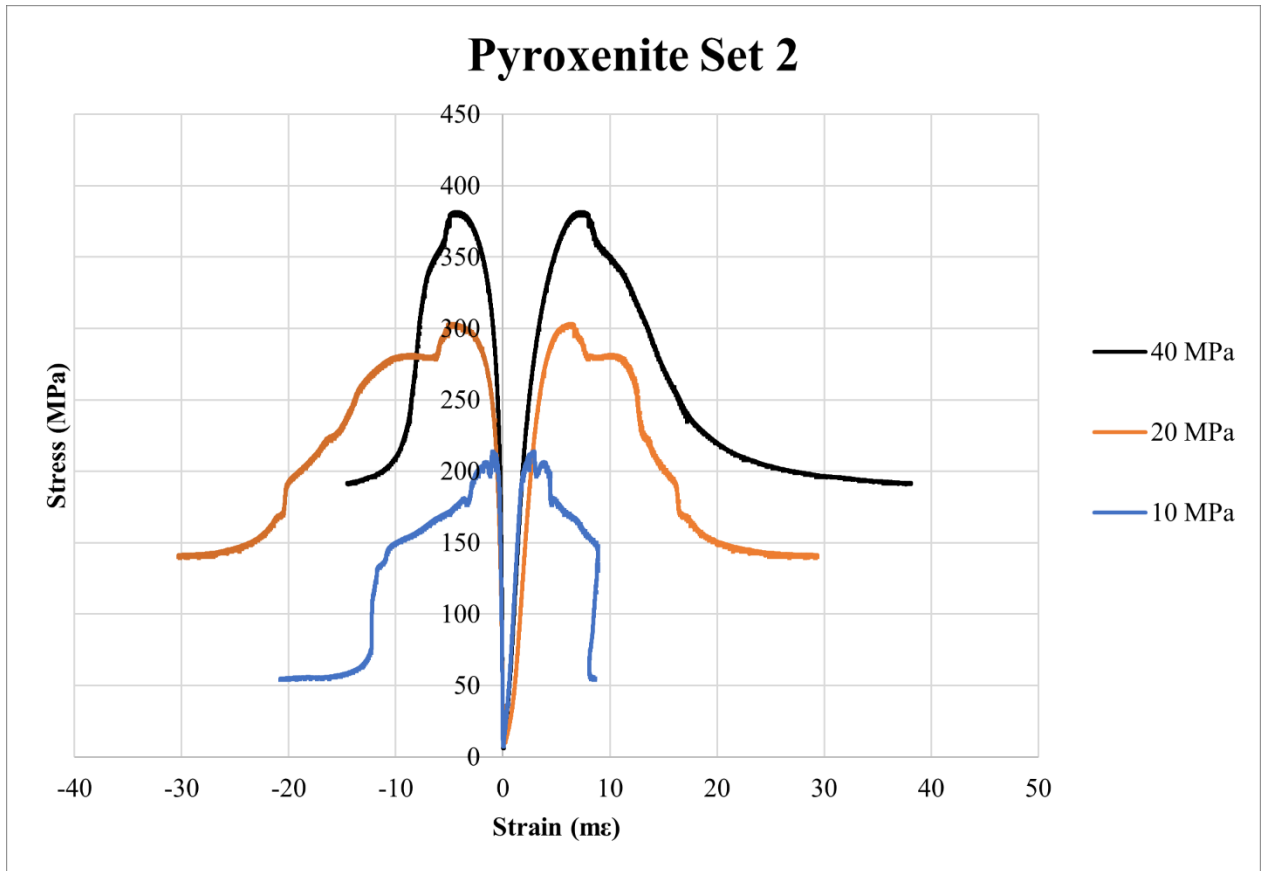


Figure A- 1 Stress-strain curves for pyroxenite set 2

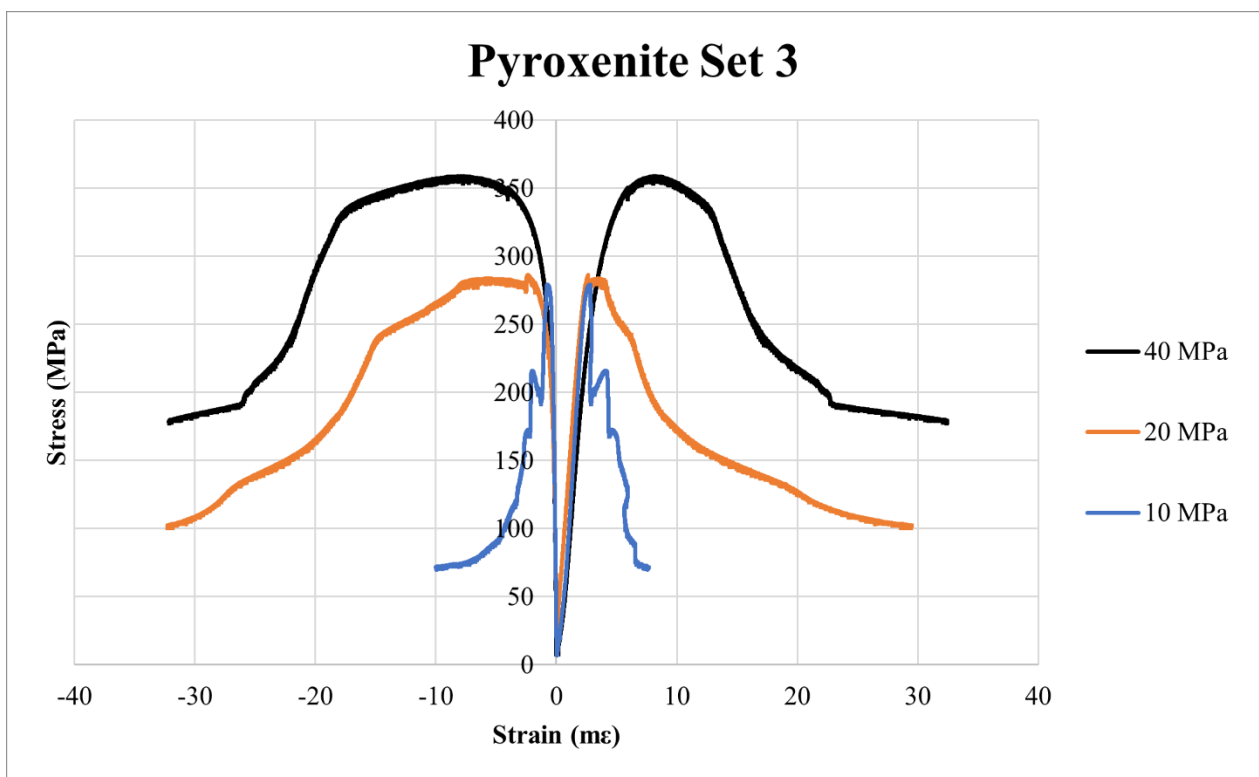


Figure A- 2 Stress-strain curves for pyroxenite set 3

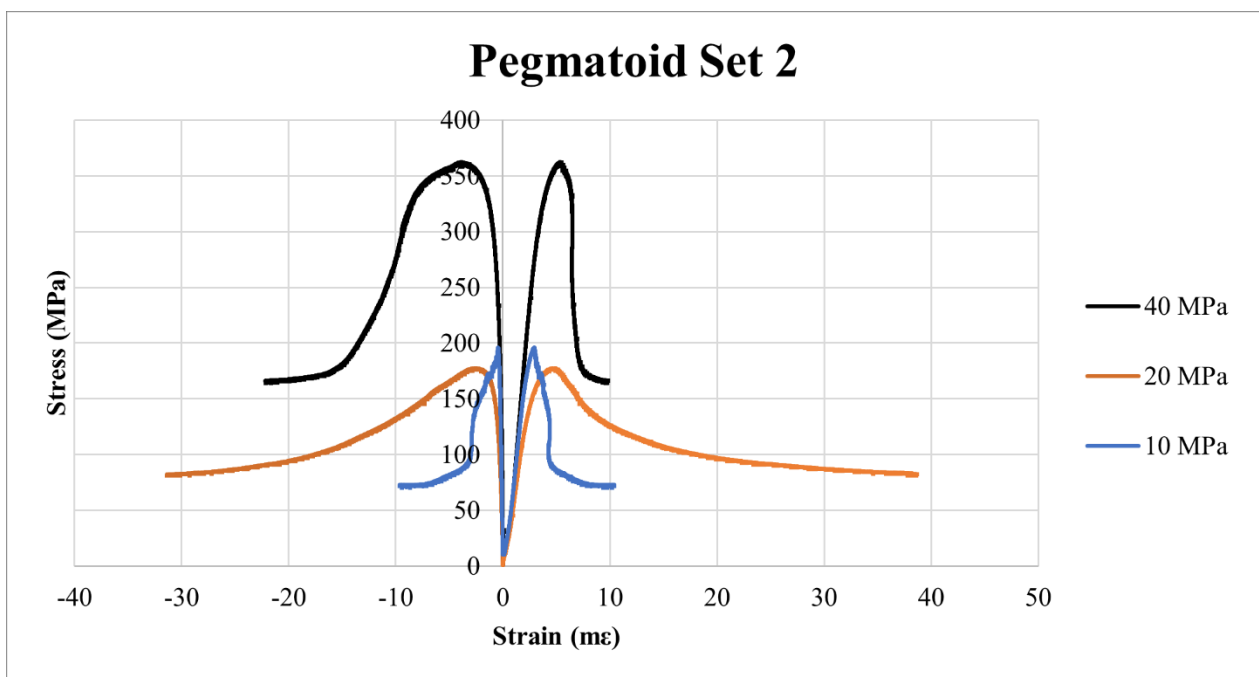


Figure A- 3 Stress-strain curves for pegmatoid set 2

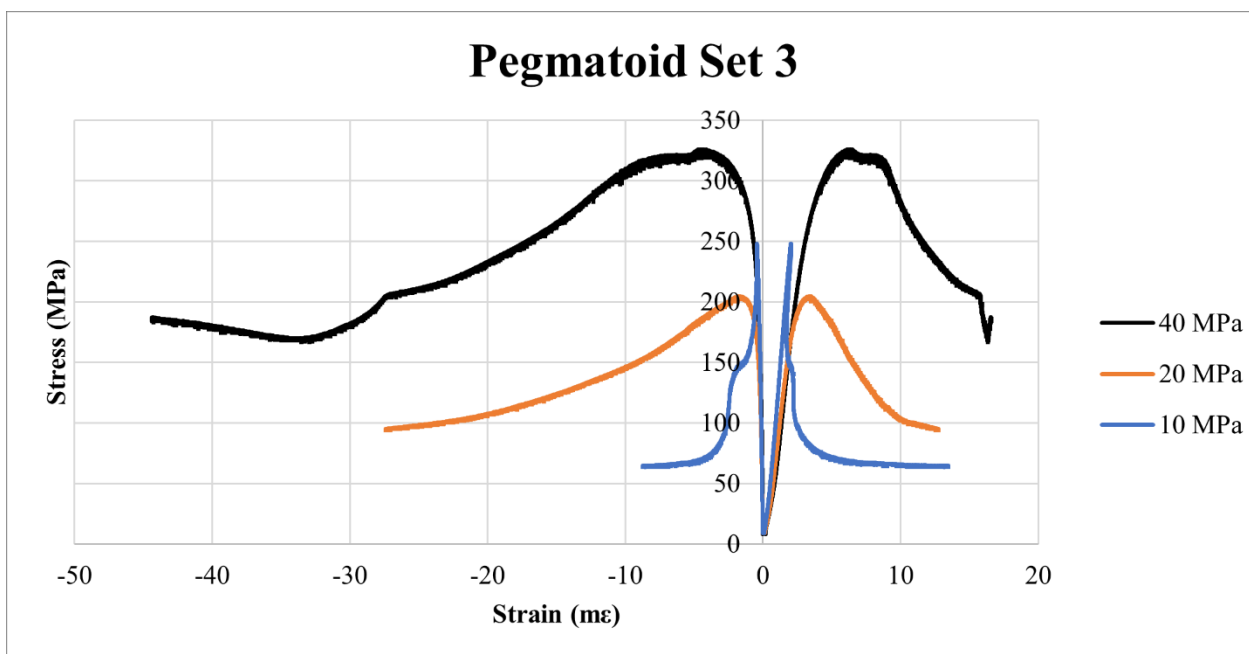


Figure A- 4 Stress-strain curves for pegmatoid set 3

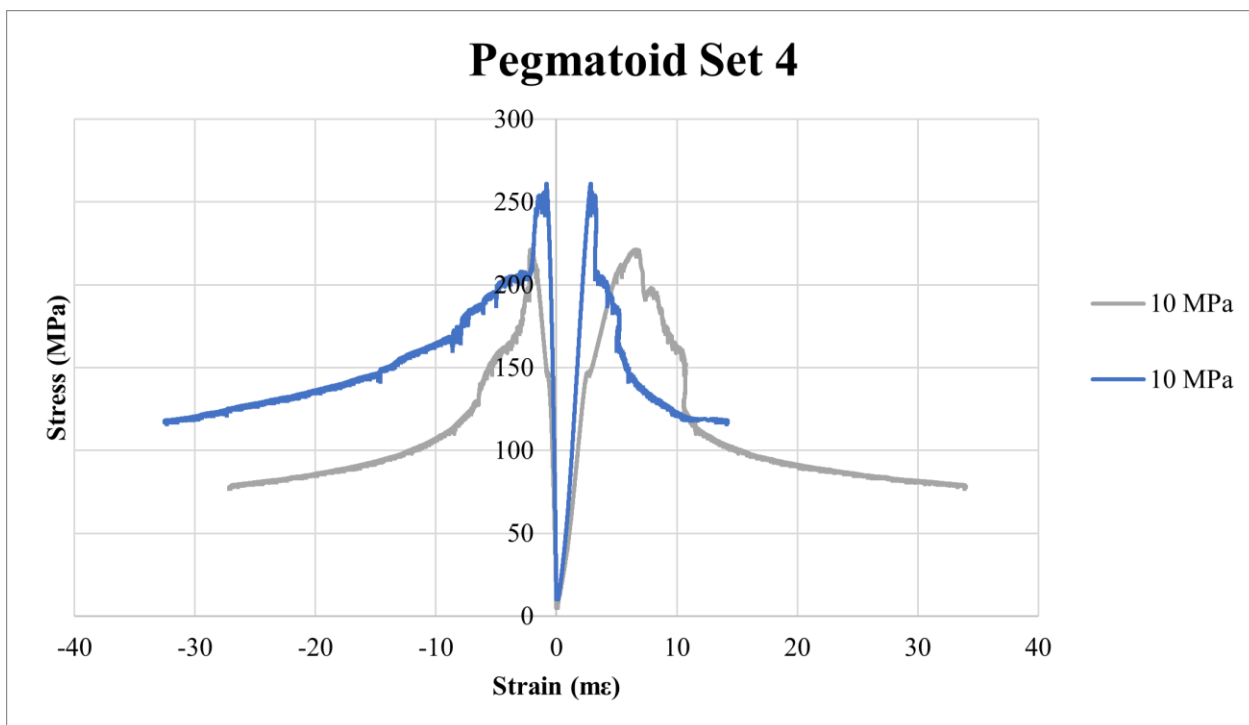


Figure A- 5 Stress-strain curves for pegmatoid set 4

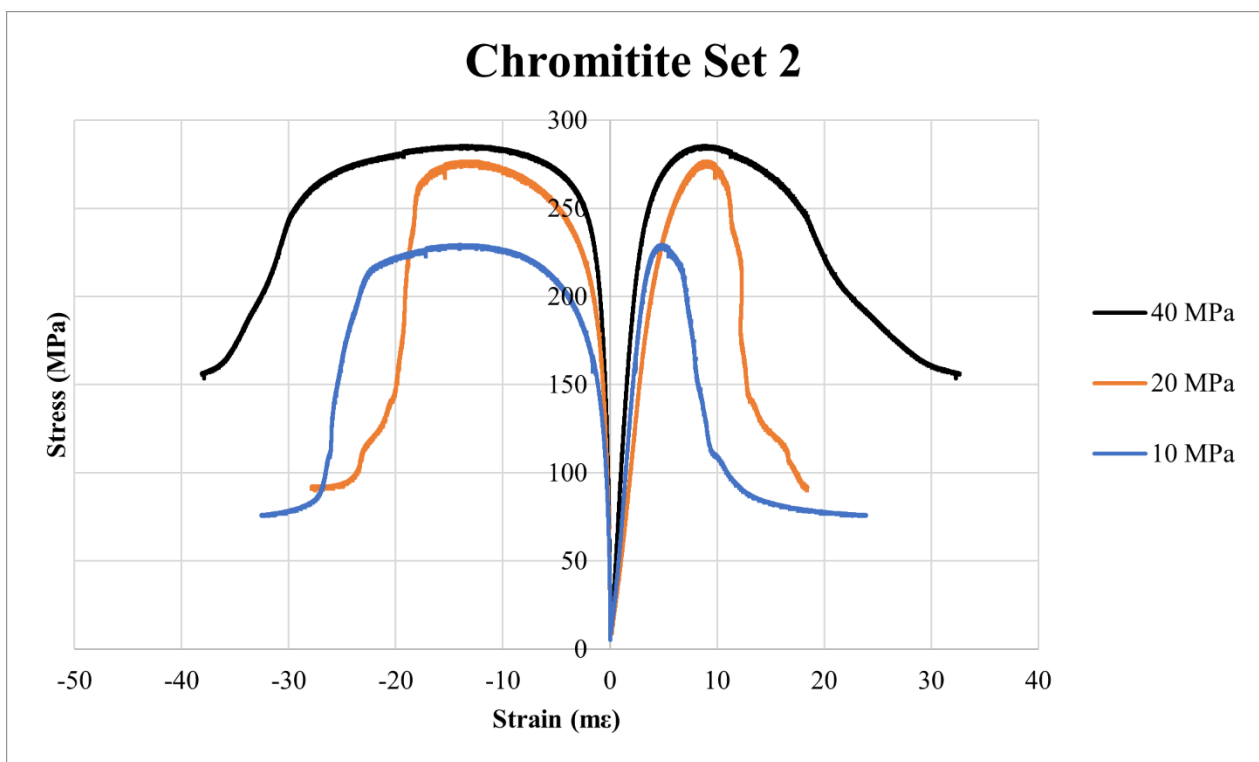


Figure A- 6 Stress-strain curves for chromitite set 2

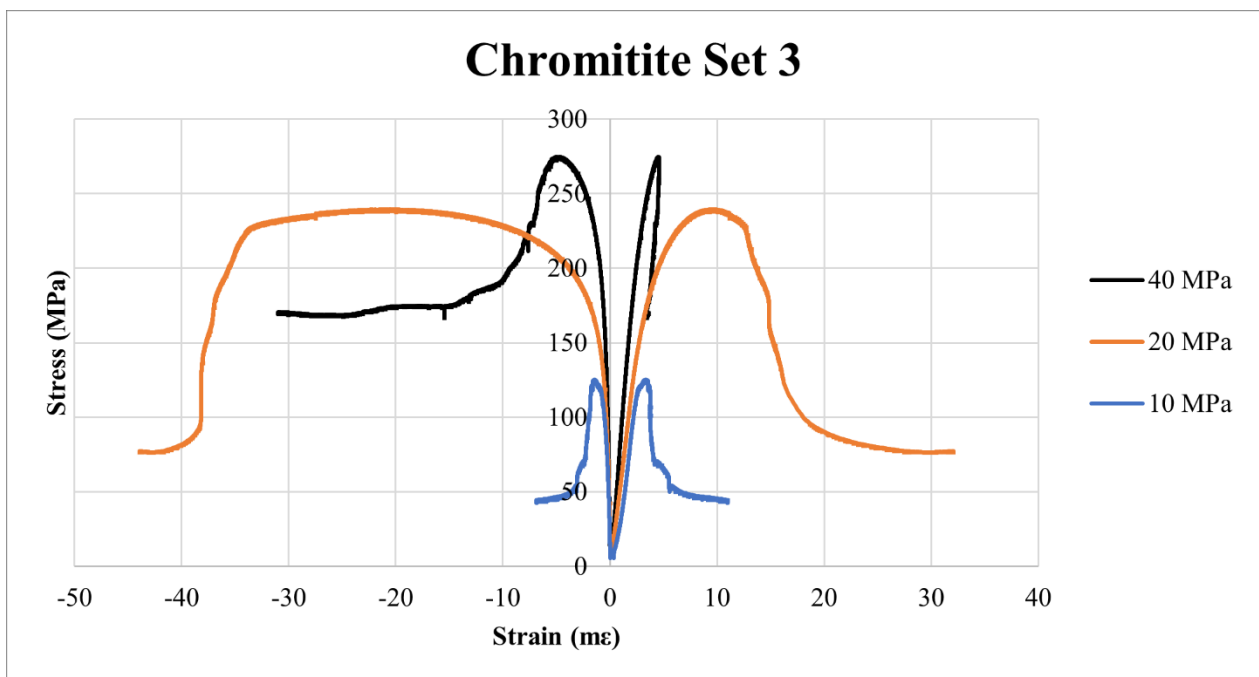


Figure A- 7 Stress-strain curve for chromitite set 3

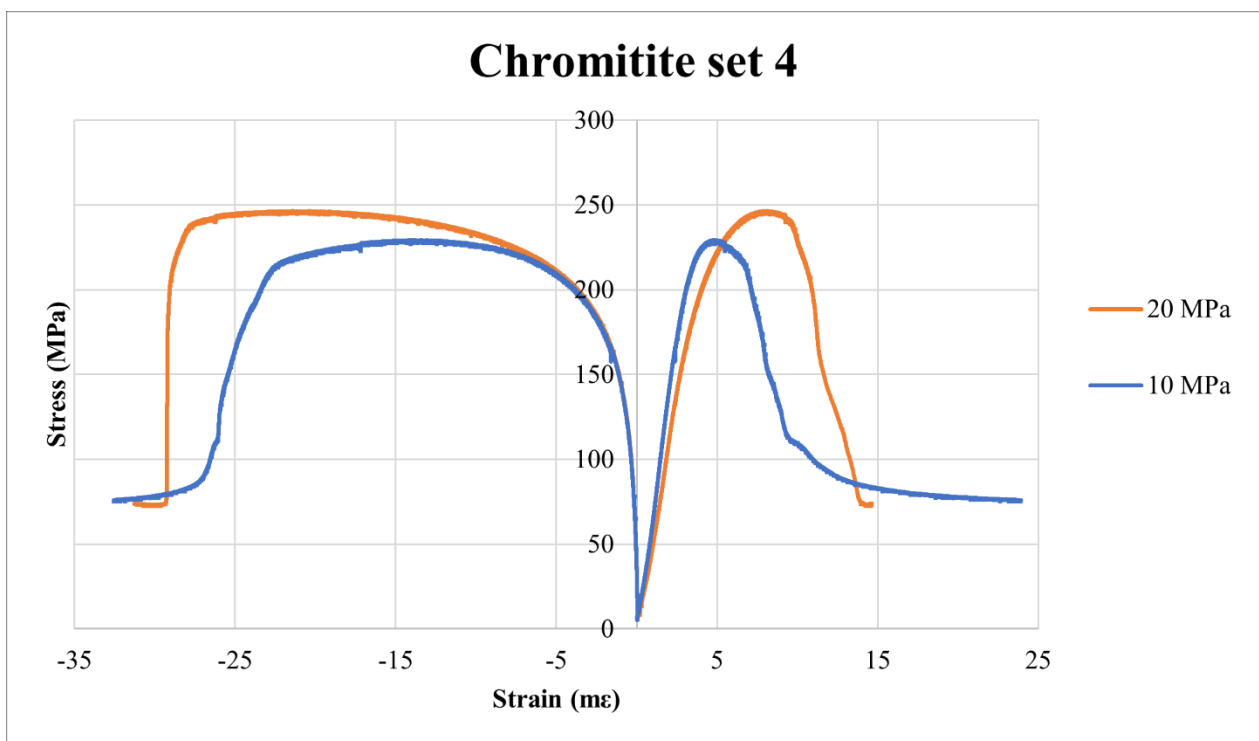


Figure A- 8 Stress-strain curve for chromitite set 4

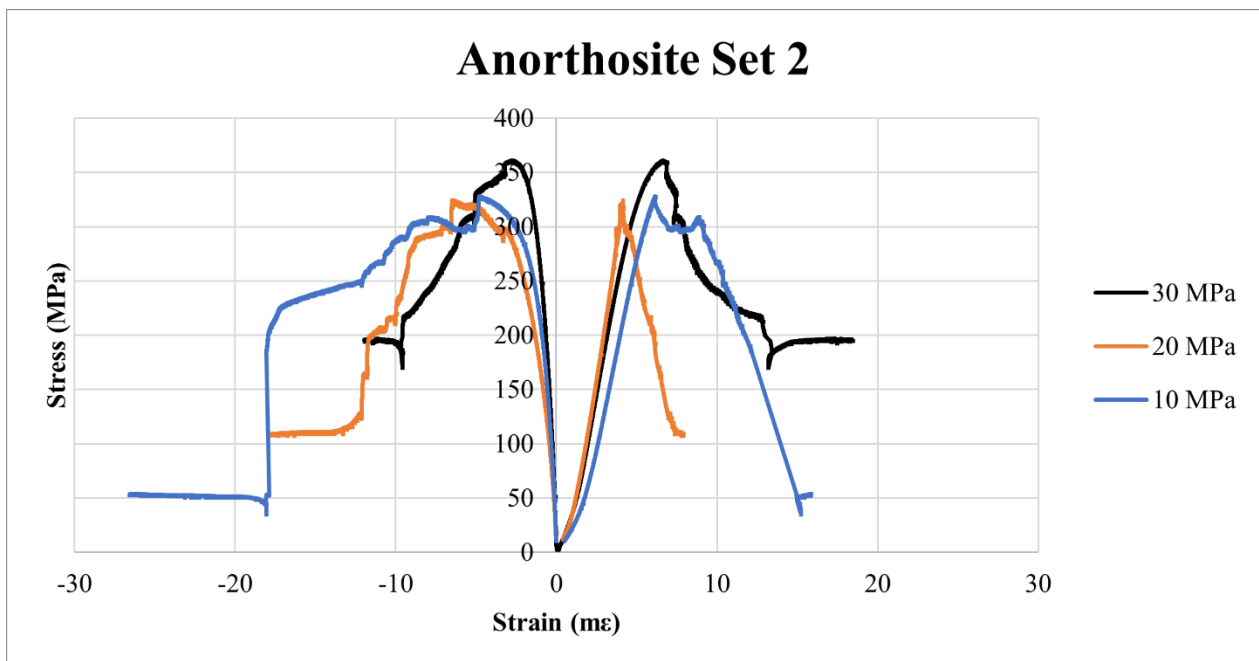


Figure A- 9 Stress-strain for anorthosite set 2

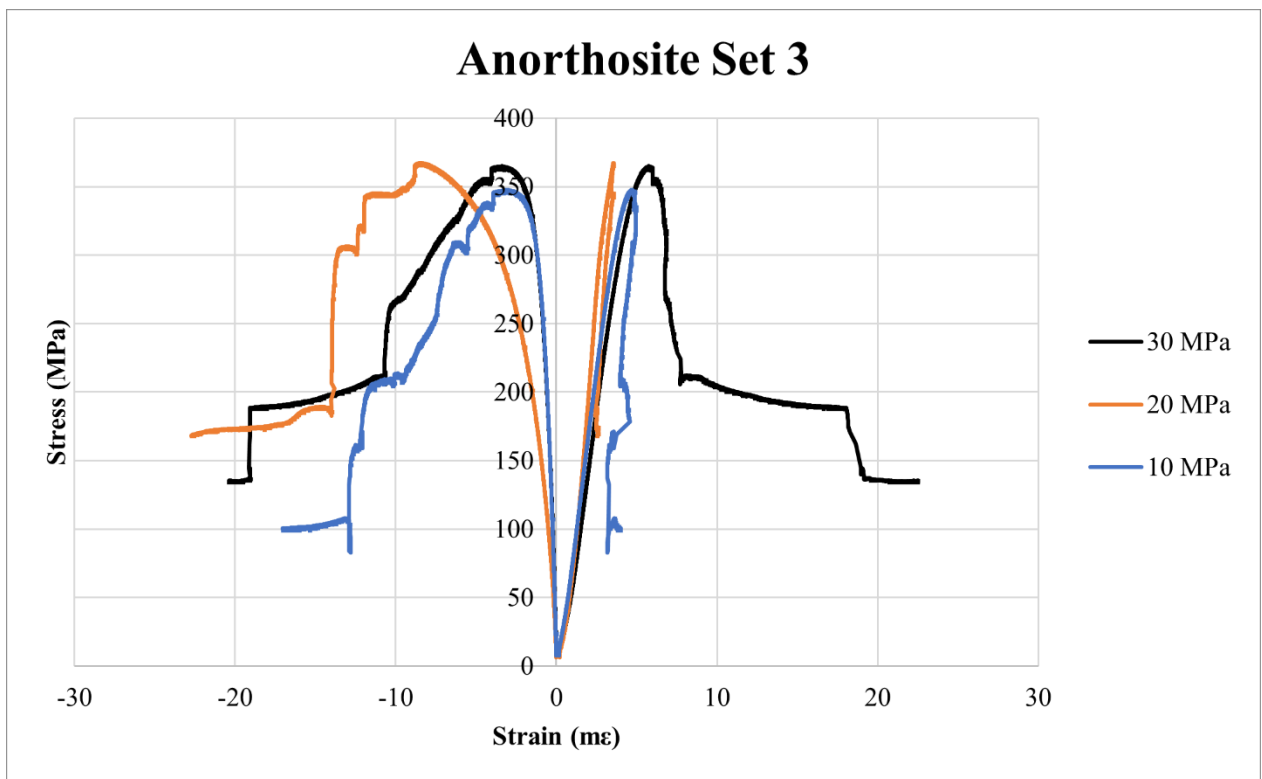


Figure A- 10 Stress-strain for anorthosite set 3

Appendix A2- Elastic constants

Table A- 1 Effect of confinement on the elastic constants

	Elastic Modulus					Poisson's ratio				
	0	10	20	30	40	0	10	20	30	40
Rock type										
Anorthosite	79.29	82.65	83.23	85.05	-	0.43	0.41	0.42	0.41	-
Chromitite	73.98	71.00	73.23	-	78.86	0.33	0.32	0.32	-	0.31
Pegmatoid	57.16	72.27	83.26	-	94.72	0.24	0.23	0.23	-	0.23
Pyroxenite	106.16	122.64	115.46	-	126.72	0.23	0.25	0.26	-	0.26

APPENDIX B

Appendix B1- Dilation angles obtained from triaxial tests

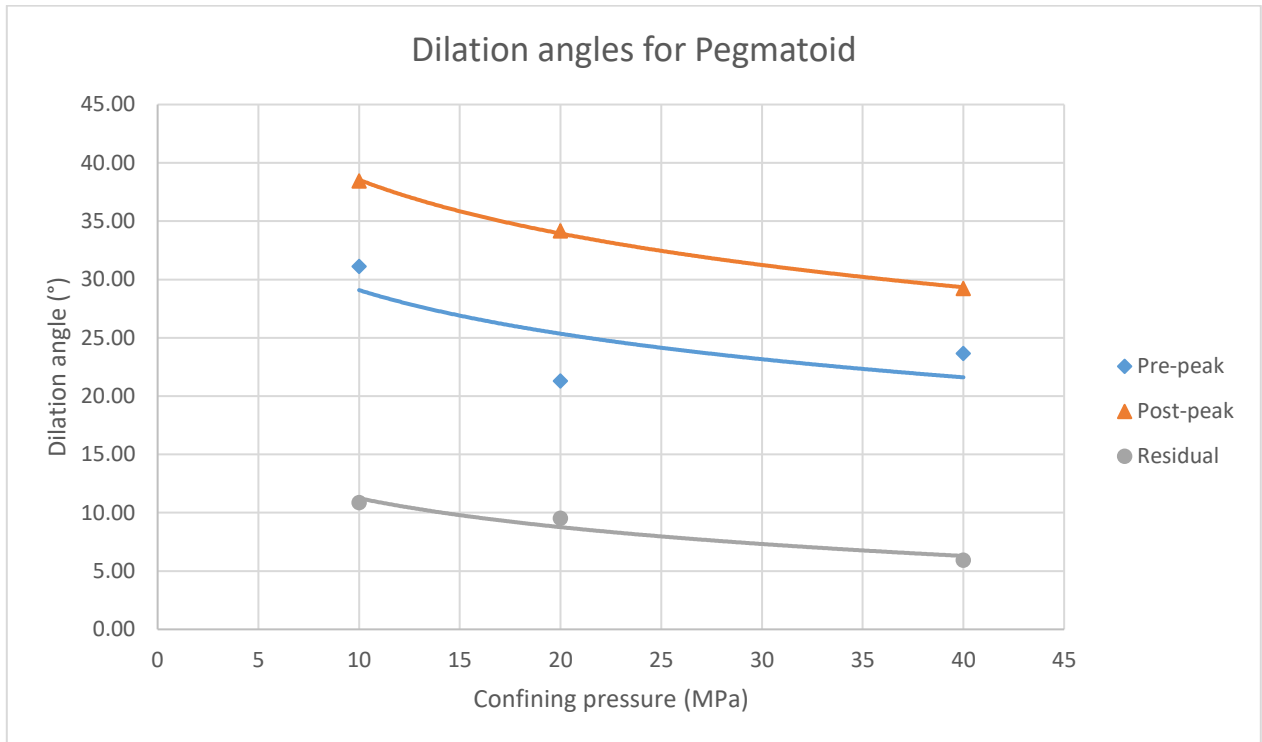


Figure B- 1 Plot of dilation angles vs confining pressure for pegmatoid

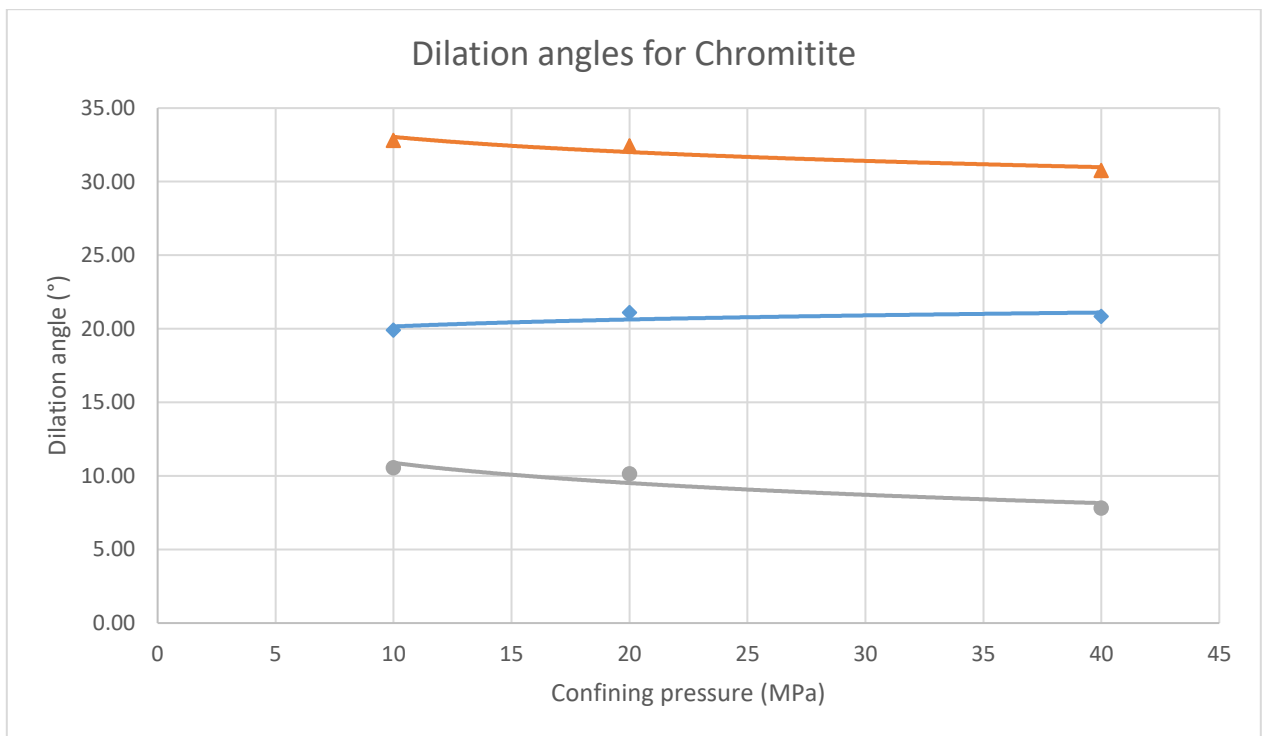


Figure B- 2 Plot of dilation angles vs confining pressure for chromitite

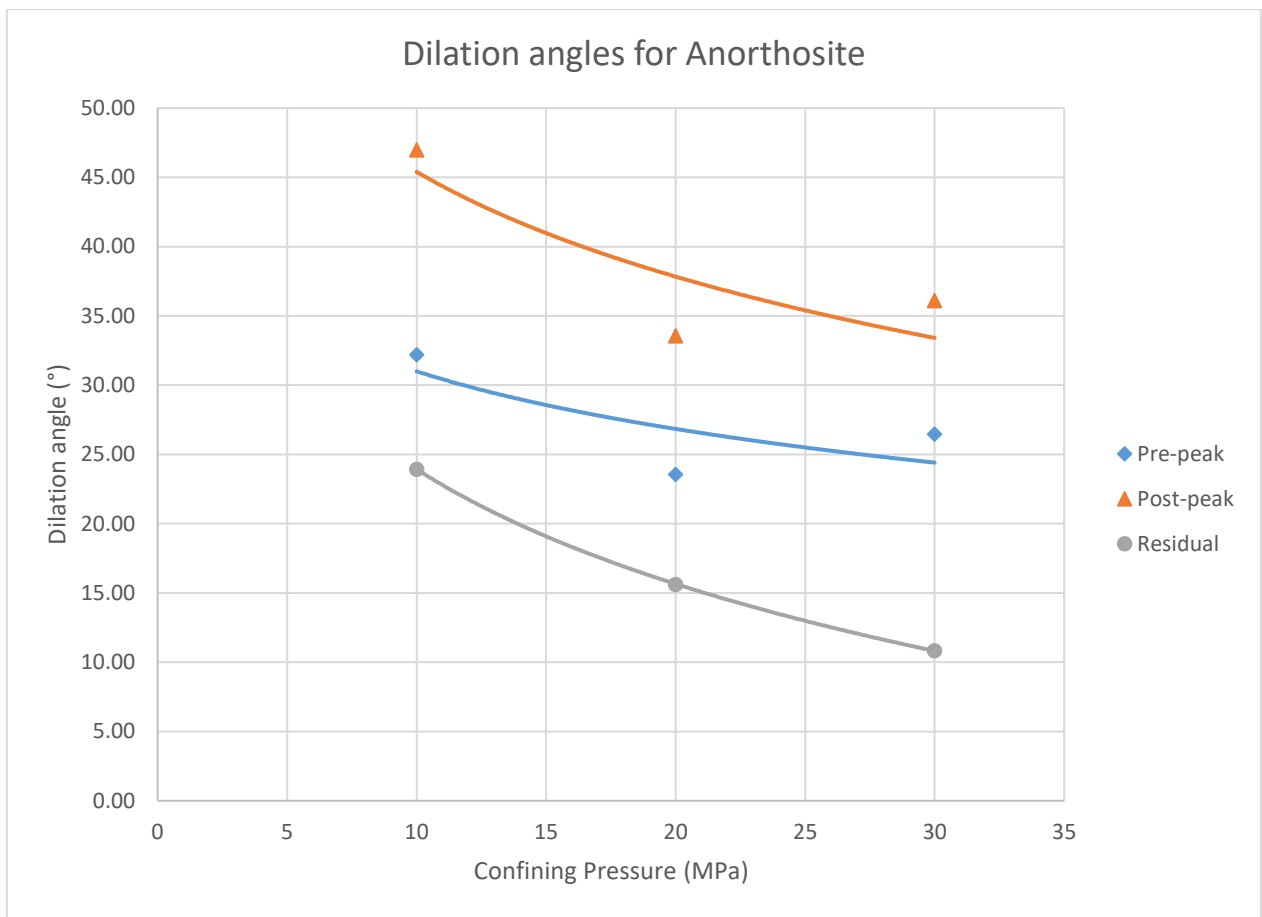


Figure B- 3 Plot of dilation angles vs confining pressure for anorthosite

APPENDIX C

Appendix C1- Mohr's circles used to estimate post-peak parameters

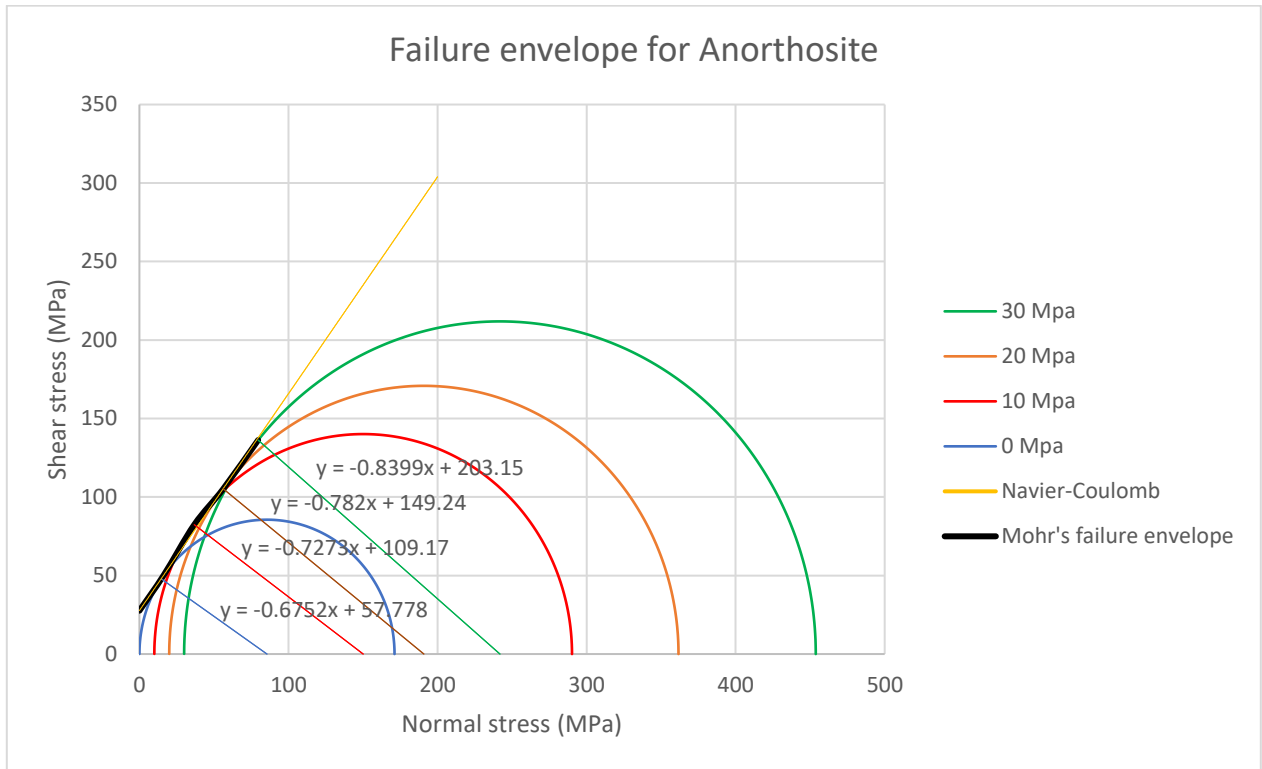


Figure C- 1 Failure envelopes for anorthosite

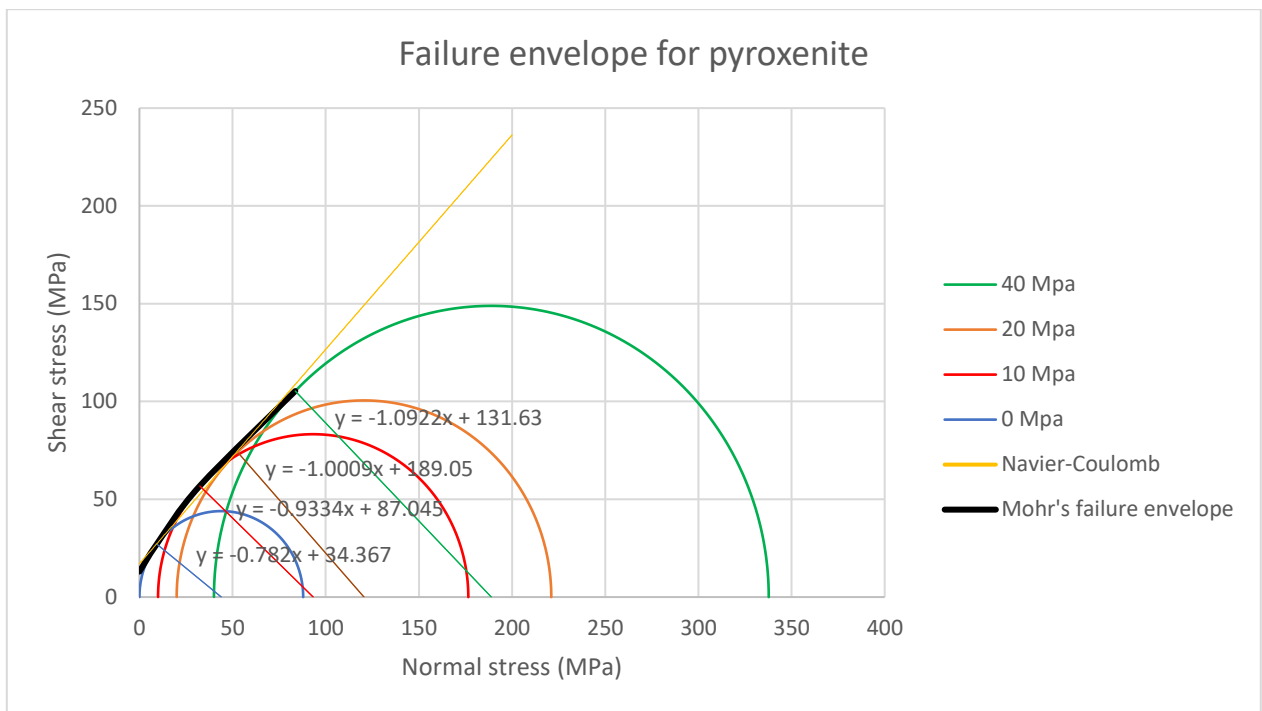


Figure C- 2 Failure envelopes for pyroxenite

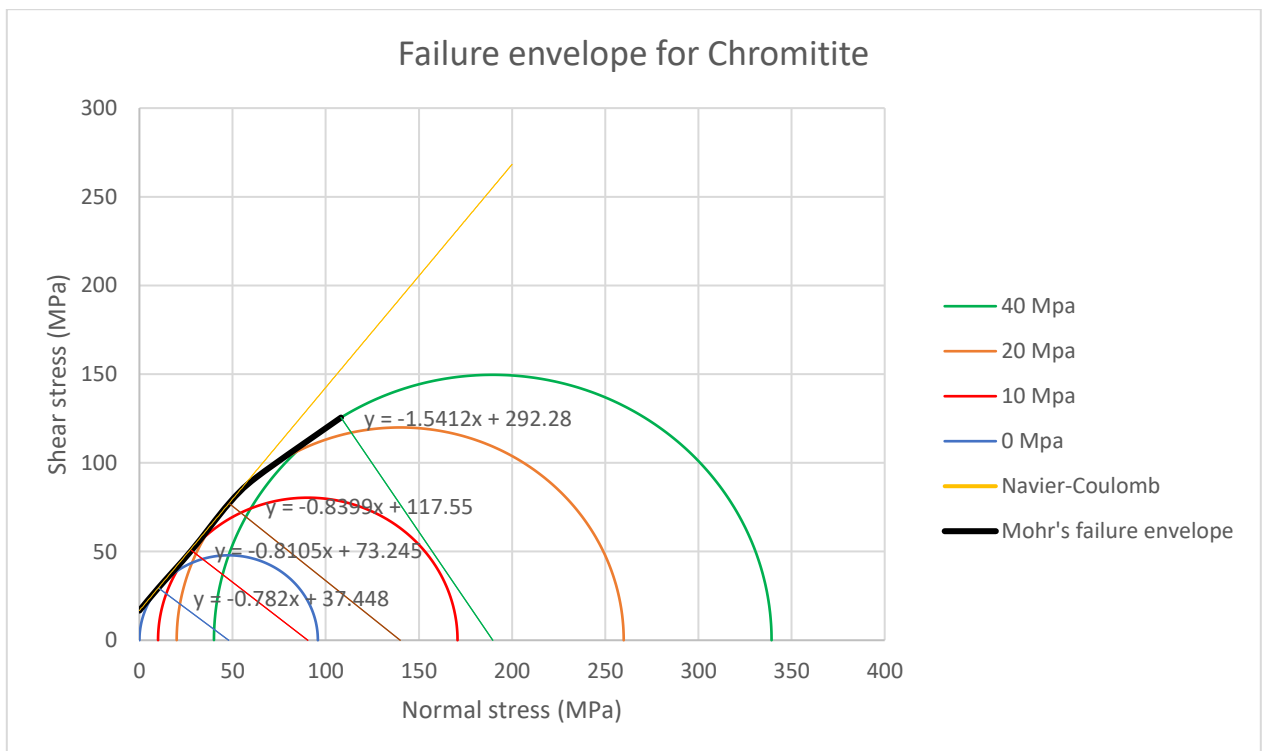


Figure C- 3 Failure envelopes for chromitite

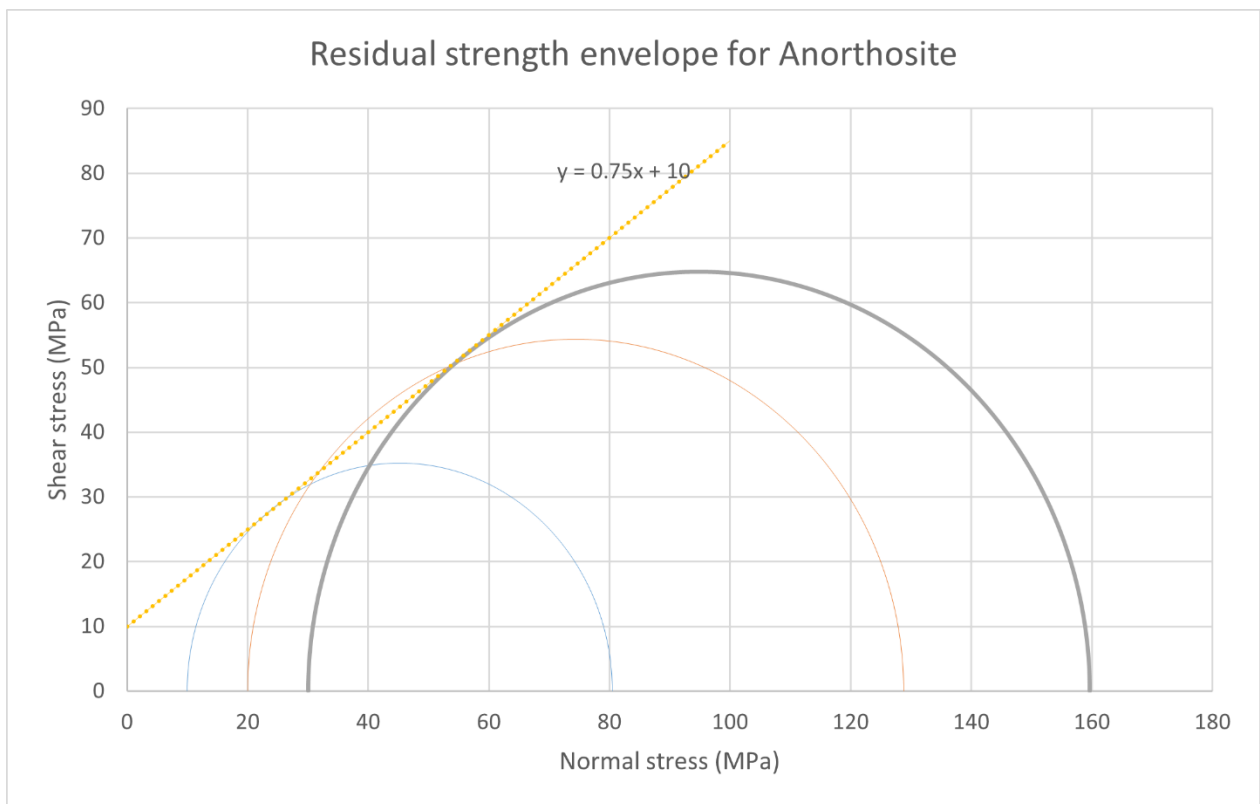


Figure C- 4 Estimation of residual cohesion and internal friction angle for anorthosite

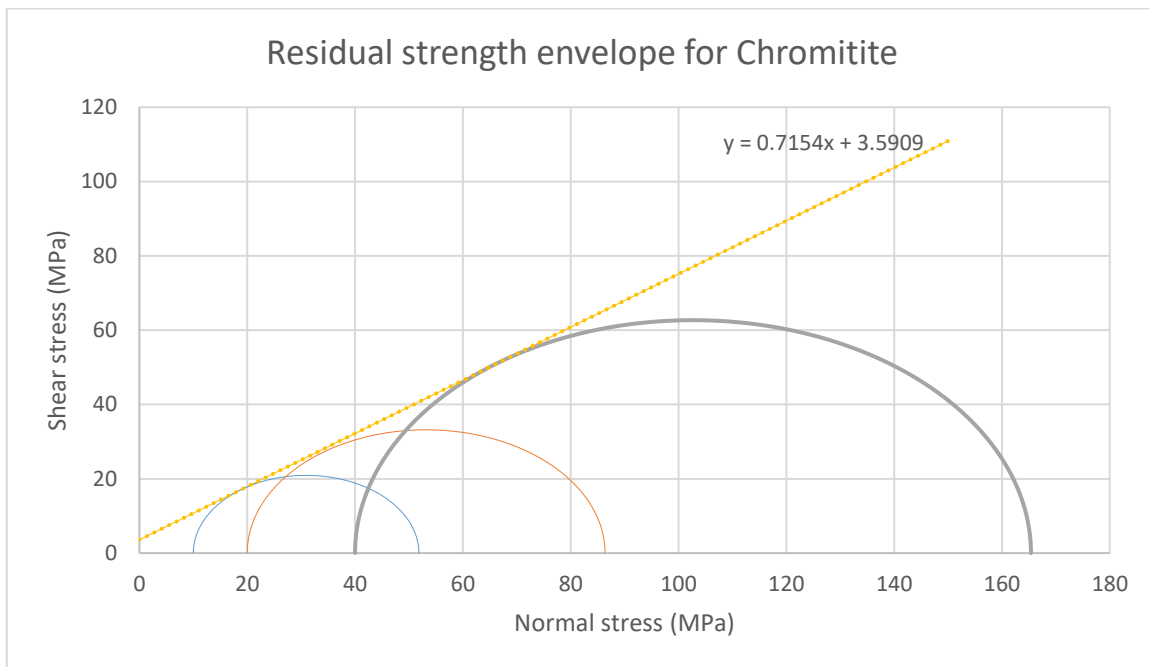


Figure C- 5 Estimation of residual cohesion and internal friction angle for chromitite

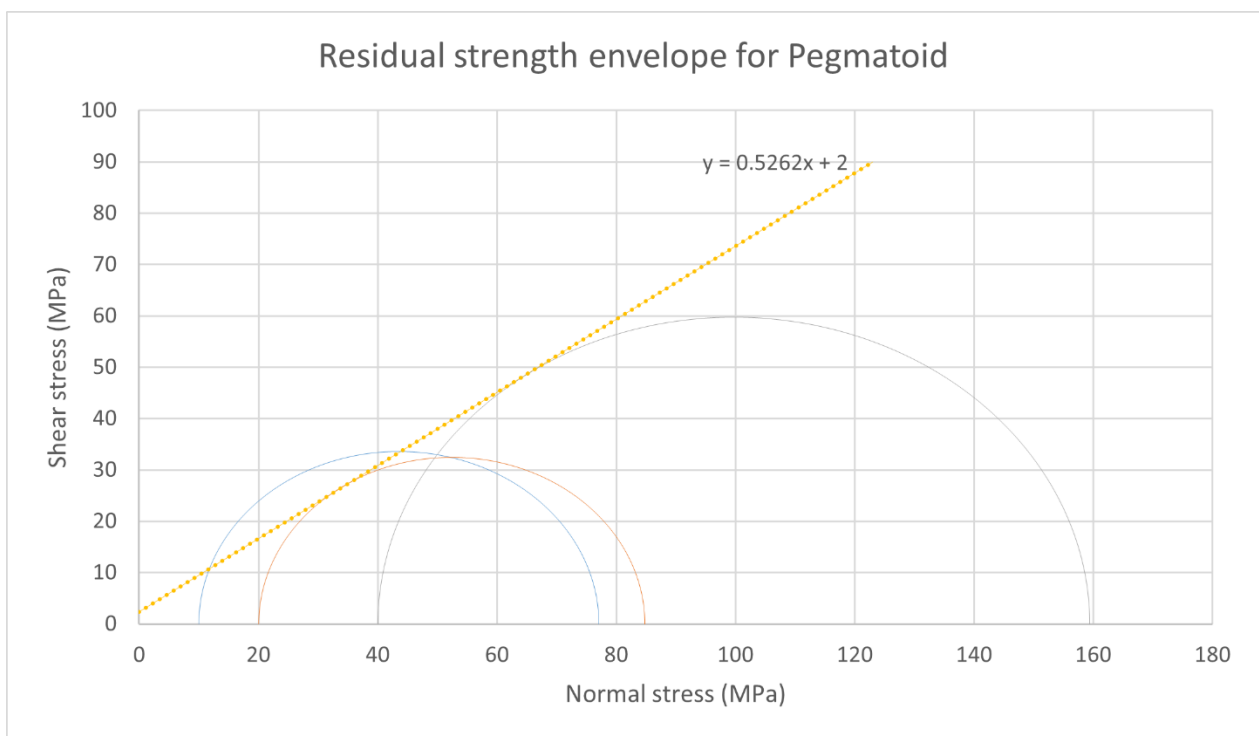


Figure C- 6 Estimation of residual cohesion and internal friction angle for Pegmatoid

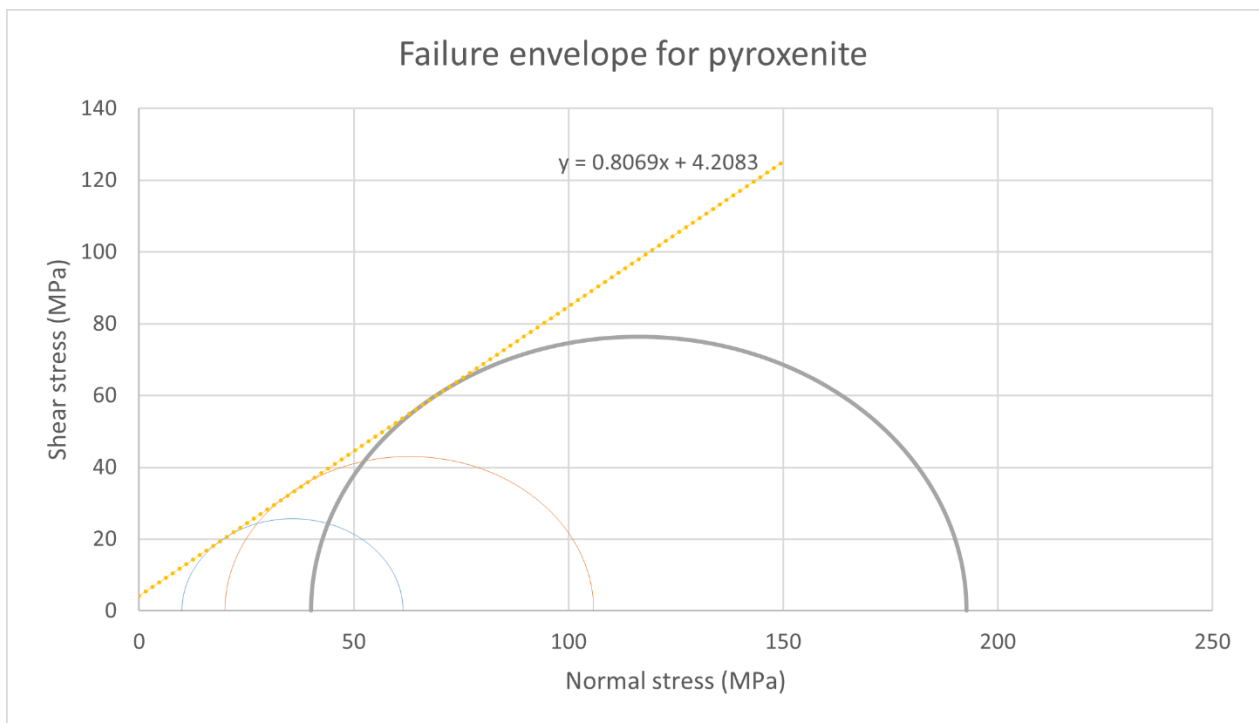


Figure C- 7 Estimation of residual cohesion and internal friction angle for pyroxenite

APPENDIX D

Appendix D1- Triaxial model results

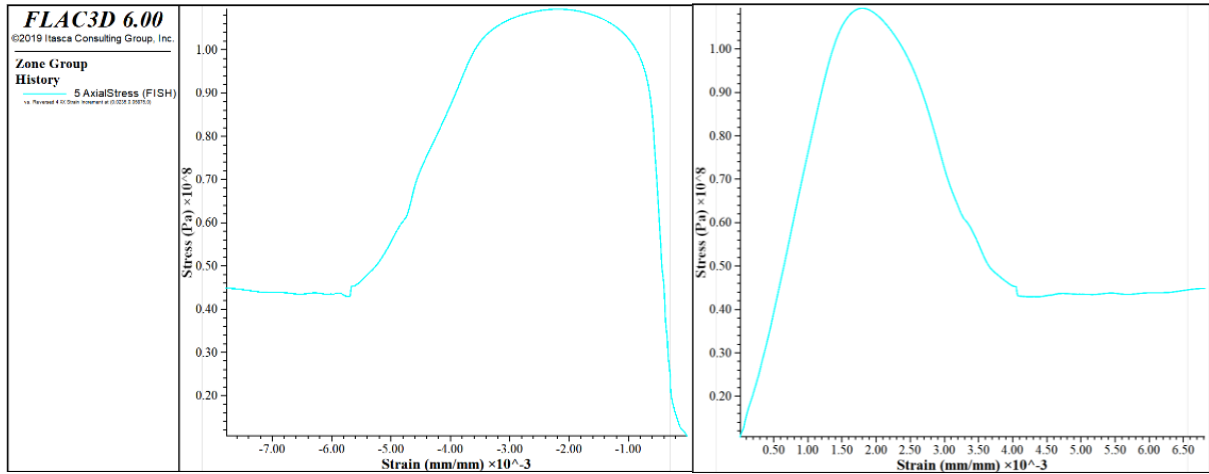


Figure D- 1 Simulated stress-strain curves for chromitite

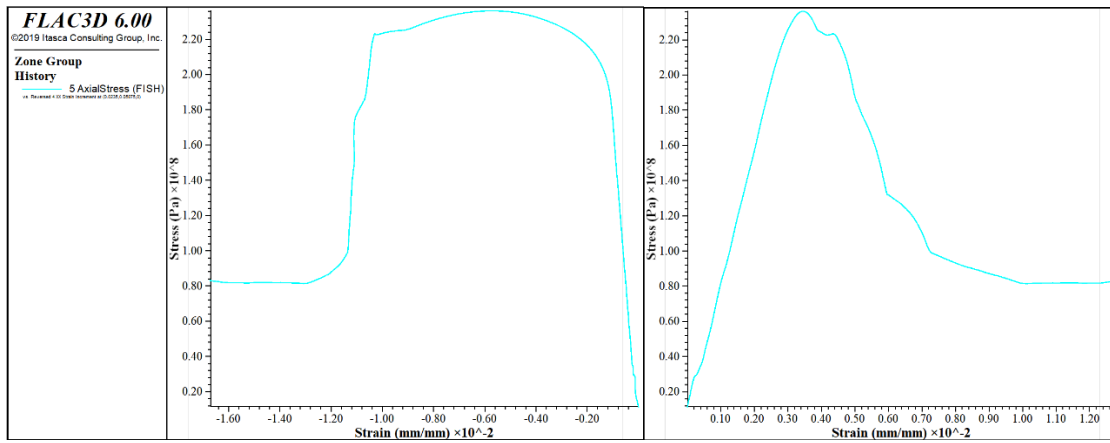


Figure D- 2 Simulated stress-strain curve for anorthosite

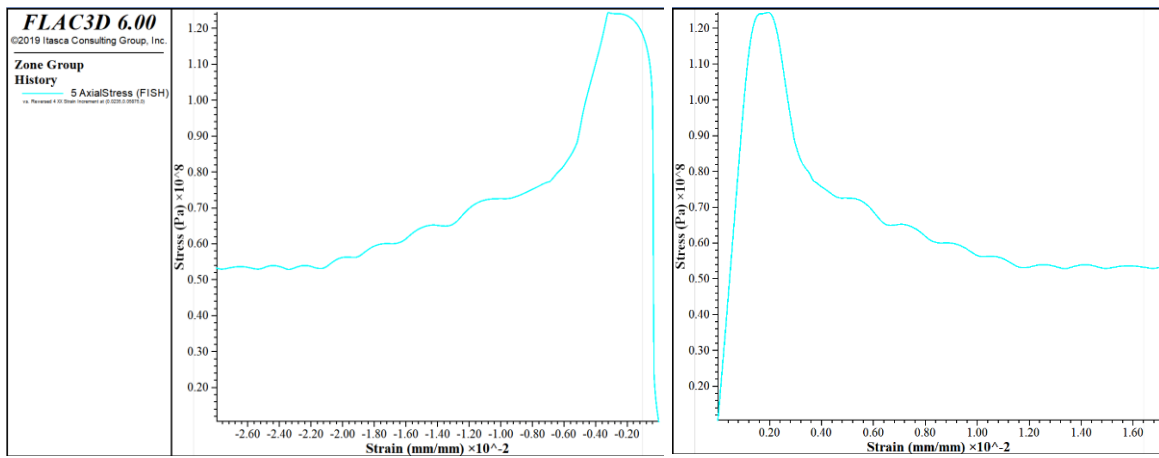


Figure D- 3 Simulated stress-strain curve for pegmatoid

Appendix D2- Triaxial model data file

Data file for Triaxial test

```
model new

zone create cylinder size 4 1 4 point 0 (0,0,0) point 1 (0.0235,0,0) ...
point 2 (0,0.1175,0) point 3 (0,0,0.0235)

zone reflect dip 0 origin (0,0,0)

zone reflect dip 90 dip-direction 270 origin (0,0,0)

; assign group name

zone group 'Specimen=pyroxenite' range position-y 0 0.1175

; assign face names

zone face skin slot "Specimen"

; Assign constitutive models and properties

zone cmodel assign strain-softening

zone property density 3224 bulk 65.53e9 shear 43.15e9

zone property cohesion 27e6 friction 47.38 tension 7.82e5 dilation 13.34

zone property table-friction 'fri' table-cohesion 'coh' table-dilation 'dil'

table 'fri' add (0, 47.38) (.001, 47.38) (0.014, 38.9) (1, 38.9)

table 'coh' add (0, 27e6) (.001, 27e6) (0.014, 13e6) (1, 13e6)

table 'dil' add (0, 34.1) (.001, 47) (0.014, 13.34)

; Boundary Conditions

fish define conf

conf = -10e6

end
```

```

@conf

zone face apply velocity-normal 0 range group 'pyroxeniteSouth'

fish define get_ad

    ad1 = zone.near(0,0.1175,0)

    ad2 = zone.near(0.0235,0.05875,0)

end

@get_ad

;initial conitions

;zone initialize stress xx [conf] yy [conf] zz [conf]

zone initialize stress xx = [conf] yy = [conf] zz = [conf]

zone face apply velocity-normal -1e-6 range group 'pyroxeniteNorth'

; Histories

history interval 1

zone history displacement-y position (0,0.1175,0)

zone history displacement-x position (0.0235,0.05875,0)

zone history strain-increment-y position (0,0.1175,0)

zone history strain-increment-x position (0.0235,0.05875,0)

; FISH to store gridpoints on y=0 surface 'South'

fish define FindSurface

    global surface = map

```

```

loop foreach local gp gp.list
    if gp.isgroup(gp,'pyroxeniteNorth') then
        surface(gp.id(gp)) = gp
    endif
endloop
end

@FindSurface

; FISH to track AxialStress on y=0.1175 surface 'South'

fish define AxialStress

    local str = 0

    loop foreach local gp surface
        str = str + gp.force.unbal.y(gp)
    endloop

    AxialStress = str / ( math.pi * 0.0235 ^ 2 ) ; cylinder radius = 23.5 mm

end

fish history @AxialStress

;model step 5000

model step 1000

; Plot of plastic region as zones with strain > 0.2

fish define TagPlasticZones

    loop foreach local zp zone.list
        if zone.prop(zp,'strain-shear-plastic') > 0.01
            zone.group(zp) = 'Yield'
        else

```

```

        zone.group(zp) = 'Other'
    endif
endloop
end

@TagPlasticZones
model save 'Pyroxenite'

```

Appendix D3- Simple pillar model with interfaces

Data file for Simple pillar model with interfaces

```

model new

call 'plotview1.f3dat'

call 'plotview shear1.f3dat'

fish define setup_variables
    global file_root = 'cp'
    global stagename = 'none'
end

@setup_variables

```

zone create brick size 20 20 20 point 0 (0,0,0) point 1 (2,0,0) ...
 point 2 (0,1,0) point 3 (0,0,0.2)
 zone create brick size 20 20 20 point 0 (0,0,0.2) point 1 (2,0,0.2) ...
 point 2 (0,1,0.2) point 3 (0,0,0.6)
 zone create brick size 20 20 20 point 0 (0,0,0.6) point 1 (2,0,0.6) ...
 point 2 (0,1,0.6) point 3 (0,0,0.8)
 zone create brick size 20 20 20 point 0 (0,0,0.8) point 1 (2,0,0.8) ...
 point 2 (0,1,0.8) point 3 (0,0,0.91)
 zone create brick size 36 16 24 point 0 (-0.2,-0.2,0.91) point 1 (2,-0.2,0.91) ...
 point 2 (-0.2,1,0.91) point 3 (-0.2,-0.2,1.91)
 zone create brick size 36 16 24 point 0 (-0.2,-0.2,-1) point 1 (2,-0.2,-1) ...
 point 2 (-0.2,1,-1) point 3 (-0.2,-0.2,0)
 zone attach by-face range position-z 0
 zone attach by-face range position-z 0.91

;assigning groups

zone group 'Pillar=Anorthosite' range position-z -1 0.2
 zone group 'Pillar=pegmatoid' range position-z 0.2 0.6
 zone group 'Pillar=Chromitite' range position-z 0.6 0.8
 zone group 'Pillar=pyroxenite' range position-z 0.8 1.91
 zone face skin slot "Pillar"

zone group 'hist=slice' range position-z 0.445 0.465 ; group used to calculate stress and strain

; Assign constitutive models and properties

zone cmodel assign strain-softening

zone property density 2749 bulk 67.86e9 shear 9.97e9 range group 'anorthosite'

zone property cohesion 12.8e6 friction 41.4 tension 2.08e5 dilation 10.4 range group 'anorthosite'

zone property table-friction 'fri_an' table-cohesion 'coh_an' range group 'anorthosite'

table 'fri_an' add (0, 36.9) (0.005,36.9) (.02, 36.9) (1, 36.9)

table 'coh_an' add (0, 12.8e6) (.005, 12.8e6) (.02, 10e6) (1, 0)

zone property density 3968 bulk 26.08e9 shear 10e9 range group 'chromitite'

zone property cohesion 6.6e6 friction 44.1 tension 0.72e5 dilation 11 range group 'chromitite'

zone property table-friction 'fri_cr' table-cohesion 'coh_cr' range group 'chromitite'

table 'fri_cr' add (0, 35.6) (0.005, 35.6) (.02, 35.6) (1, 35.6)

table 'coh_cr' add (0, 8.8e6) (0.005, 8.8e6) (.02, 3.4e6) (1, 0)

zone property density 3224 bulk 23.52e9 shear 15.49e9 range group 'pyroxenite'

zone property cohesion 10.5e6 friction 42.39 tension 1.48e5 dilation 9.5 range group 'pyroxenite'

zone property table-friction 'fri_px' table-cohesion 'coh_px' range group 'pyroxenite'

table 'fri' add (0, 35) (0.005, 35) (.02, 35) (1, 35)

table 'coh' add (0, 14.2e6) (0.005, 14.2e6) (.02, 13e6) (1, 0)

```

zone property density 2801 bulk 13.14e9 shear 8.27e9 range group 'pegmatoid'
zone property cohesion 4.9e6 friction 32.8 tension 2.8e5 dilation 8.2 range group
'pegmatoid'
zone property table-friction 'fri_pg' table-cohesion 'coh_pg' range group 'pegmatoid'
table 'fri_pg' add (0, 30) (0.005, 30) (.02, 30) (1, 30)
table 'coh_pg' add (0, 4.9e6) (0.005, 4.9e6) (.02, 2e6) (1, 8e4)

;interface
zone separate by-face new-side group 'interface1' range group 'pegmatoid' group
'anorthosite'
zone interface 1 create by-face range group 'Interface1'
zone interface 1 node property stiffness-normal 1e12 stiffness-shear 1e12 ...
friction 27 cohesion 1e6

zone separate by-face new-side group 'interface2' range group 'chromitite' group
'pegmatoid'
zone interface 2 create by-face range group 'Interface2'
zone interface 2 node property stiffness-normal 1e12 stiffness-shear 1e12 ...
friction 24 cohesion 1e6

```

```
zone separate by-face new-side group 'interface3' range group 'pyroxenite' group  
'chromitite'
```

```
zone interface 3 create by-face range group 'Interface3'
```

```
zone interface 3 node property stiffness-normal 1e12 stiffness-shear 1e12 ...  
friction 29 cohesion 1e6
```

```
; Assign Boundary Conditions
```

```
zone face apply velocity-normal 0 range position-x 2
```

```
zone face apply velocity-normal 0 range position-x -0.2
```

```
zone face apply velocity-normal 0 range position-y 1
```

```
zone face apply velocity-normal 0 range position-y -0.2
```

```
zone face apply velocity-normal 0 range position-z -1
```

```
zone face apply velocity-normal -1e-6 range position-z 1.91
```

```
fish define setzones
```

```
;
```

```
minz = 0.445
```

```
maxz = 0.465
```

```
global slicezones = map
```

```
loop foreach local zp zone.list
```

```
if zone.pos.z(zp) > minz & zone.pos.z(zp) < maxz
```

```
slicezones(zone.id(zp)) = zp
```

```
        endif
    end_loop
;
end
@setzones
```

```
fish define load

    sum = 0

    nzon = 0

    loop foreach local zp slicezones

        sum = sum - zone.stress.zz(zp)

        nzon = nzon + 1

    end_loop

    load = sum / nzon

end
```

```
fish define setzp
;

global gps = map

loop foreach local p zone.list

    if zone.isgroup(p,'slice')

        gps(zone.id(p)) = p

    endif

end
```

```

    end_loop

;

end

@setzp


fish define strain

a = 0


loop foreach local p gps

a = a - zone.strain.inc.zz(p)


end_loop

strain = a / 400

end


; Histories

history interval 1

fish history name 'load' @load

fish history name "strain" @strain

fish define doplots

;

fname_szz = file_root + '_szz_' + stagename + '.png'

fname_pss = file_root + '_pss_' + stagename + '.png'

;

```

```

command

;

plot 'SZZ' export bitmap filename [fname_szz]

plot 'pss' export bitmap filename [fname_pss]

;

end_command

;

end

fish define runmodel

;

totsteps = 40000          ; total number of steps to run analysis

noutputs = 20             ; number of outputs

stepsperstage = int(totsteps/noutputs)

;

loop istage (1,noutputs)

;

  stagename = string(istage)

  command

    model step [stepsperstage]

  end_command

;

  doplots

;

end_loop

```

```
;  
end  
@runmodel  
model save 'crush pillar'
```

Call file plotview1.f3dat

plot create 'szz'

plot clear

plot active on

plot background 'white'

plot outline active on width 2 color 'black'

plot legend active on heading color 'black' copyright color 'black' ...

 placement left size 25,50 ...

 step active off ...

 time-real active off ...

 time-model active off ...

 title-customer active off ...

 view-info active off

plot title-job active off

plot title active off

plot view projection parallel magnification 1.365 ...

 center (0.9,0.4,0.455) eye (4.8545,0.4,0.455) roll 0.000 ...

 clip-front -1e+10 clip-back 1e+10

plot item create zone active on ...

 contour stress quantity zz stress off log off ...

 method constant null fluid off mechanical off thermal off ...

 ramp rainbow ...

 minimum automatic maximum 0 interval 2.5e+06 ...

reversed off above automatic below automatic ...

polygons fill on outline active on width 1 color 'black' ...

outline-transparency 80 lighting on offset 0.5 2 selected-only off selected-highlight
off ...

null-faces-only off ...

map axis xyz translate (0,0,0) scale (1,1,1) ...

deformation-factor active off ...

hide-null mechanical on thermal off fluid off ...

transparency 0 ...

cut active off ...

clip active off ...

legend active on ...

title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

map active on size 44 family 'Arial' style normal color 'black' ...

hide-null active off ...

deformation-factor active on size 44 family 'Arial' style normal color 'black' ...

contour labels active on size 44 family 'Arial' style normal color 'black' ...

labels-maximum 20 ...

method active on size 44 family 'Arial' style normal color 'black'

Call file plotview shear1.f3dat

plot create 'pss'

plot clear

plot active on

plot background 'white'

plot outline active on width 2 color 'black'

plot legend active on heading color 'black' copyright color 'black' ...

 placement left size 25,50 ...

 step active off ...

 time-real active off ...

 time-model active off ...

 title-customer active off ...

 view-info active off

plot title-job active off

plot title active off

plot view projection parallel magnification 1.365 ...

 center (0.9,0.4,0.455) eye (4.8545,0.4,0.455) roll 0.000 ...

 clip-front -1e+10 clip-back 1e+10

plot item create zone active on ...

 contour property log off name 'strain-shear-plastic' ...

 method constant null fluid off mechanical off thermal off ...

 ramp rainbow ...

 minimum 0 maximum 0.012 interval 1e-05 ...

reversed off above automatic below automatic ...

polygons fill on outline active on width 1 color 'black' ...

outline-transparency 80 lighting on offset 0.5 2 selected-only off selected-highlight
off ...

null-faces-only off ...

map axis xyz translate (0,0,0) scale (1,1,1) ...

deformation-factor active off ...

hide-null mechanical on thermal off fluid off ...

transparency 0 ...

cut active off ...

clip active off ...

legend active on ...

title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

map active on size 44 family 'Arial' style normal color 'black' ...

hide-null active off ...

deformation-factor active on size 44 family 'Arial' style normal color 'black' ...

contour labels active on size 44 family 'Arial' style normal color 'black' ...

labels-maximum 20 ...

method active on size 44 family 'Arial' style normal color 'black'

Appendix D4- Crush pillar model with siding (Scenario 1)

Data file for crush pillar with siding (Scenario 1)

model new

call 'plotview1.f3dat'

call 'plotview shear1.f3dat'

fish define setup_variables

global file_root = 'cp'

global stagename = 'none'

end

@setup_variables

;Grid Generation

zone create brick size 15 6 5 point 0 (0,0,0) point 1 (12,0,0) ...

point 2 (0,5.5,0) point 3 (0,0,3) ratio 0.9 1 0.9

zone create brick size 20 6 10 point 0 (12,0,0) point 1 (30,0,0) ...

point 2 (12,5.5,0) point 3 (12,0,4.8) ratio 1.1 1 0.9

zone create brick size 10 6 6 point 0 (0,0,3) point 1 (10.7,0,3) ...

point 2 (0,5.5,3) point 3 (0,0,4.8) ratio 0.9 1 0.9

zone create brick size 20 20 20 point 0 (14,0.75,4.8) point 1 (16,0.75,4.8) ...

point 2 (14,4.75,4.8) point 3 (14,0.75,5.71)

zone create brick size 20 6 10 point 0 (0,0,5.71) point 1 (15,0,5.71) ...
 point 2 (0,5.5,5.71) point 3 (0,0,10.71) ratio 0.9 1 1.1
 zone create brick size 20 6 10 point 0 (15,0,5.71) point 1 (30,0,5.71) ...
 point 2 (15,5.5,5.71) point 3 (15,0,10.71) ratio 1.1 1 1.1
 zone attach by-face range position-z 3
 zone attach by-face range position-z 4.8
 zone attach by-face range position-z 5.71
 zone attach by-face range position-x 15
 zone attach by-face range position-x 12
 ;assigning groups
 zone group 'Pillar=Anorthosite' range position-z 0 5
 zone group 'Pillar=pegmatoid' range position-z 5 5.2
 zone group 'Pillar=Chromitite' range position-z 5.2 5.6
 zone group 'Pillar=pyroxenite' range position-z 5.6 10.71
 zone face skin slot "Pillar"
 zone group 'hist=slice' range position-z 5.175 5.335 ; group used to calculate stress and strain
 ;Section through pillar
 zone group 'Section=a' range position-y 0 2.75
 zone group 'Section=b' range position-y 2.75 5.5
 ; Assign constitutive models and properties
 zone cmodel assign strain-softening
 zone property density 2749 bulk 67.86e9 shear 9.97e9 range group 'anorthosite'

zone property cohesion 12.8e6 friction 41.4 tension 2.08e5 dilation 10.4 range group
'anorthosite'

zone property table-friction 'fri_an' table-cohesion 'coh_an' range group 'anorthosite'

table 'fri_an' add (0, 36.9) (0.005,36.9) (.02, 36.9) (1, 36.9)

table 'coh_an' add (0, 12.8e6) (.005, 12.8e6) (.02, 10e6) (1, 0)

zone property density 3968 bulk 26.08e9 shear 10e9 range group 'chromitite'

zone property cohesion 6.6e6 friction 44.1 tension 0.72e5 dilation 11 range group
'chromitite'

zone property table-friction 'fri_cr' table-cohesion 'coh_cr' range group 'chromitite'

table 'fri_cr' add (0, 35.6) (0.005, 35.6) (.02, 35.6) (1, 35.6)

table 'coh_cr' add (0, 8.8e6) (0.005, 8.8e6) (.02, 3.4e6) (1, 0)

zone property density 3224 bulk 23.52e9 shear 15.49e9 range group 'pyroxenite'

zone property cohesion 10.5e6 friction 42.39 tension 1.48e5 dilation 9.5 range group
'pyroxenite'

zone property table-friction 'fri_px' table-cohesion 'coh_px' range group 'pyroxenite'

table 'fri' add (0, 35) (0.005, 35) (.02, 35) (1, 35)

table 'coh' add (0, 14.2e6) (0.005, 14.2e6) (.02, 13e6) (1, 0)

zone property density 2801 bulk 13.14e9 shear 8.27e9 range group 'pegmatoid'

zone property cohesion 4.9e6 friction 32.8 tension 2.8e5 dilation 8.2 range group
'pegmatoid'

zone property table-friction 'fri_pg' table-cohesion 'coh_pg' range group 'pegmatoid'

table 'fri_pg' add (0, 30) (0.005, 30) (.02, 30) (1, 30)

table 'coh_pg' add (0, 4.9e6) (0.005, 4.9e6) (.02, 2e6) (1, 8e4)

;interface

zone separate by-face new-side group 'interface1' range group 'pegmatoid' group
'anorthosite'

zone interface 1 create by-face range group 'Interface1'

zone interface 1 node property stiffness-normal 1e12 stiffness-shear 1e12 ...

friction 30 cohesion 1e6

zone separate by-face new-side group 'interface2' range group 'chromitite' group
'pegmatoid'

zone interface 2 create by-face range group 'Interface2'

zone interface 2 node property stiffness-normal 1e12 stiffness-shear 1e12 ...

friction 33 cohesion 1e6

zone separate by-face new-side group 'interface3' range group 'pyroxenite' group
'chromitite'

zone interface 3 create by-face range group 'Interface3'

zone interface 3 node property stiffness-normal 1e12 stiffness-shear 1e12 ...

friction 35 cohesion 1e6

```

; Assign Boundary Conditions

zone face apply velocity-normal 0 range position-x 0
zone face apply velocity-normal 0 range position-x 30
zone face apply velocity-normal 0 range position-y 0
zone face apply velocity-normal 0 range position-y 5.5
zone face apply velocity-normal 0 range position-z 0
zone face apply velocity-normal -1e-6 range position-z 10.71

fish define setzones

;

minz = 5.175

maxz = 5.335

global slicezones = map

loop foreach local zp zone.list

    if zone.pos.z(zp) > minz & zone.pos.z(zp) < maxz

        slicezones(zone.id(zp)) = zp

    endif

end_loop

;

end

@setzones

fish define load

```

```

sum = 0

nzon = 0

loop foreach local zp slicezones
    sum = sum - zone.stress.zz(zp)
    nzon = nzon + 1
end_loop

load = sum / nzon

end

```

```

fish define setzp
;

global gps = map

loop foreach local p zone.list
    if zone.isgroup(p,'slice')
        gps(zone.id(p)) = p
    endif
end_loop

;

end

@setzp

```

```

fish define strain

a = 0

```

```
loop foreach local p gps
```

```
a = a - zone.strain.inc.zz(p)
```

```
end_loop
```

```
strain = a / 1600
```

```
end
```

```
; Histories
```

```
history interval 1
```

```
fish history name 'load' @load
```

```
fish history name "strain" @strain
```

```
fish define doplots
```

```
;
```

```
fname_szz = file_root + '_szz_' + stagename + '.png'
```

```
fname_pss = file_root + '_pss_' + stagename + '.png'
```

```
;
```

```
command
```

```
;
```

```
plot 'SZZ' export bitmap filename [fname_szz]
```

```
plot 'pss' export bitmap filename [fname_pss]
```

```
;
```

```
end_command
```

```
;
```

end

fish define runmodel

;

totsteps = 500000 ; total number of steps to run analysis

noutputs = 80 ; number of outputs

stepsperstage = int(totsteps/noutputs)

;

loop istage (1,noutputs)

;

stagename = string(istage)

command

model step [stepsperstage]

end_command

;

doplots

;

end_loop

;

end

@runmodel

model save 'crush pillar with siding 1'

Data file for call plotview1

plot create 'szz'

plot clear

plot active on

plot background 'white'

plot outline active on width 2 color 'black'

plot legend active on heading color 'black' copyright color 'black' ...

 placement left size 25,50 ...

 step active off ...

 time-real active off ...

 time-model active off ...

 title-customer active off ...

 view-info active off

plot title-job active off

plot title active off

plot view projection parallel magnification 0.879886 ...

 center (14.659,3.8812,4.4357) eye (14.659,-10.386,4.4357) roll 0.000 ...

 clip-front -1e+10 clip-back 1e+10

plot item create chart-history active off ...

 history 'load' name "load load (FISH)" style line reversed-x off reversed-y off ...

 style-line width 2 style solid color 'seagreen' ...

alias "" ...

vs 'strain' reversed off ...

begin 0 end 0 skip 0 ...

axis-x log off minimum auto maximum auto inside on ...

label "Strain (mm/mm)" exponent show on value -16 auto on ...

axis-y log off minimum auto maximum auto inside on ...

label "Stress (Pa)" exponent show on value -6 auto on ...

title active off ...

labels active on size 20 family 'Times New Roman' style bold ...

marks active on size 18 family 'Times New Roman' style normal ...

chart width 2 box off color 'black' position 1 1 size 96 97 ...

grid active off ...

transparency 0 ...

legend active on ...

title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

series active on size 44 family 'Arial' style normal color 'black' ...

range off ...

vs active on text "" size 44 family 'Arial' style normal color 'black' ...

vs-range off

plot item create gridpoint-fix active off ...

color-list global on clear ...

label "Fixed in local X" active on color 'cyan' ...

label "Fixed in local Y" active on color 'seagreen' ...

label "Fixed in local Z" active on color 'red' ...

line width 1 style solid ...
 scale target 0.02 value 0.317193 automatic ...
 map axis xyz translate (0,0,0) scale (1,1,1) ...
 def-fac active off ...
 transparency 0 ...
 cut active off ...
 clip active off ...
 legend active on ...
 title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...
 line-count active off ...
 scale active on size 44 family 'Arial' style normal color 'black' ...
 map active on size 44 family 'Arial' style normal color 'black' ...
 def-fac active on size 44 family 'Arial' style normal color 'black'
 plot item create zone active on ...
 contour stress quantity zz stress off log off ...
 method constant null fluid off mechanical off thermal off ...
 ramp rainbow ...
 minimum -3e+08 maximum 0 interval 500000 ...
 reversed off above automatic below automatic ...
 polygons fill on outline active on width 1 color 'black' ...
 outline-transparency 80 lighting on offset 0.5 2 selected-only off selected-highlight
 off ...
 null-faces-only off ...
 map axis xyz translate (0,0,0) scale (1,1,1) ...

deformation-factor active off ...

hide-null mechanical on thermal off fluid off ...

transparency 0 ...

cut active off ...

clip active off ...

legend active on ...

 title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

 map active on size 44 family 'Arial' style normal color 'black' ...

 hide-null active off ...

 deformation-factor active on size 44 family 'Arial' style normal color 'black' ...

 contour labels active on size 44 family 'Arial' style normal color 'black' ...

 labels-maximum 20 ...

 method active on size 44 family 'Arial' style normal color 'black' ...

range group "Section=a" slot "Any" not

plot item create zone active off ...

 contour property log off name 'strain-shear-plastic' ...

 method constant null fluid off mechanical off thermal off ...

ramp rainbow ...

 minimum 0 maximum 0.012 interval 0.0001 ...

 reversed off above automatic below automatic ...

polygons fill on outline active on width 1 color 'black' ...

 outline-transparency 80 lighting on offset 0.5 2 selected-only off selected-highlight
off ...

 null-faces-only off ...

map axis xyz translate (0,0,0) scale (1,1,1) ...
deformation-factor active off ...
hide-null mechanical on thermal off fluid off ...
transparency 0 ...
cut active off ...
clip active off ...
legend active on ...
 title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...
 map active on size 44 family 'Arial' style normal color 'black' ...
 hide-null active off ...
 deformation-factor active on size 44 family 'Arial' style normal color 'black' ...
 contour labels active on size 44 family 'Arial' style normal color 'black' ...
 labels-maximum 20 ...
 method active on size 44 family 'Arial' style normal color 'black'

Data file for call plotview shear1

plot create 'pss'
plot clear
plot active on
plot background 'white'
plot outline active on width 2 color 'black'
plot legend active on heading color 'black' copyright color 'black' ...
 placement left size 25,50 ...
 step active off ...

time-real active off ...
time-model active off ...
title-customer active off ...
view-info active off
plot title-job active off
plot title active off
plot view projection parallel magnification 0.879886 ...
center (14.659,3.8812,4.4357) eye (14.659,-10.386,4.4357) roll 0.000 ...
clip-front -1e+10 clip-back 1e+10
plot item create chart-history active off ...
history 'load' name "load load (FISH)" style line reversed-x off reversed-y off ...
style-line width 2 style solid color 'seagreen' ...
alias "" ...
vs 'strain' reversed off ...
begin 0 end 0 skip 0 ...
axis-x log off minimum auto maximum auto inside on ...
label "Strain (mm/mm)" exponent show on value -16 auto on ...
axis-y log off minimum auto maximum auto inside on ...
label "Stress (Pa)" exponent show on value -6 auto on ...
title active off ...
labels active on size 20 family 'Times New Roman' style bold ...
marks active on size 18 family 'Times New Roman' style normal ...
chart width 2 box off color 'black' position 1 1 size 96 97 ...
grid active off ...

transparency 0 ...

legend active on ...

title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

series active on size 44 family 'Arial' style normal color 'black' ...

range off ...

vs active on text "" size 44 family 'Arial' style normal color 'black' ...

vs-range off

plot item create gridpoint-fix active off ...

color-list global on clear ...

label "Fixed in local X" active on color 'cyan' ...

label "Fixed in local Y" active on color 'seagreen' ...

label "Fixed in local Z" active on color 'red' ...

line width 1 style solid ...

scale target 0.02 value 0.317193 automatic ...

map axis xyz translate (0,0,0) scale (1,1,1) ...

def-fac active off ...

transparency 0 ...

cut active off ...

clip active off ...

legend active on ...

title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

line-count active off ...

scale active on size 44 family 'Arial' style normal color 'black' ...

map active on size 44 family 'Arial' style normal color 'black' ...

def-fac active on size 44 family 'Arial' style normal color 'black'

plot item create zone active on ...

contour property log off name 'strain-shear-plastic' ...

method constant null fluid off mechanical off thermal off ...

ramp rainbow ...

minimum 0 maximum 0.012 interval 0.0001 ...

reversed off above automatic below automatic ...

polygons fill on outline active on width 1 color 'black' ...

outline-transparency 80 lighting on offset 0.5 2 selected-only off selected-highlight off ...

null-faces-only off ...

map axis xyz translate (0,0,0) scale (1,1,1) ...

deformation-factor active off ...

hide-null mechanical on thermal off fluid off ...

transparency 0 ...

cut active off ...

clip active off ...

legend active on ...

title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

map active on size 44 family 'Arial' style normal color 'black' ...

hide-null active off ...

deformation-factor active on size 44 family 'Arial' style normal color 'black' ...

contour labels active on size 44 family 'Arial' style normal color 'black' ...

labels-maximum 20 ...

method active on size 44 family 'Arial' style normal color 'black' ...

range group "Section=a" slot "Any" not

plot item create zone active off ...

contour property log off name 'strain-shear-plastic' ...

method constant null fluid off mechanical off thermal off ...

ramp rainbow ...

minimum 0 maximum 0.012 interval 0.0001 ...

reversed off above automatic below automatic ...

polygons fill on outline active on width 1 color 'black' ...

outline-transparency 80 lighting on offset 0.5 2 selected-only off selected-highlight
off ...

null-faces-only off ...

map axis xyz translate (0,0,0) scale (1,1,1) ...

deformation-factor active off ...

hide-null mechanical on thermal off fluid off ...

transparency 0 ...

cut active off ...

clip active off ...

legend active on ...

title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

map active on size 44 family 'Arial' style normal color 'black' ...

hide-null active off ...

deformation-factor active on size 44 family 'Arial' style normal color 'black' ...

contour labels active on size 44 family 'Arial' style normal color 'black' ...

labels-maximum 20 ...

method active on size 44 family 'Arial' style normal color 'black'

Appendix D4- Crush pillar model without a siding (Scenario 2)

model new

call 'plotview2.f3dat'

call 'plotview shear2.f3dat'

fish define setup_variables

global file_root = 'cp'

global stagename = 'none'

end

@setup_variables

;GRID generation

zone create brick size 15 6 5 point 0 (0,0,0) point 1 (14,0,0) ...

point 2 (0,5.5,0) point 3 (0,0,3) ratio 0.88 1 0.9

zone create brick size 20 6 10 point 0 (14,0,0) point 1 (30,0,0) ...

point 2 (14,5.5,0) point 3 (14,0,4.8) ratio 1.1 1 0.9

zone create brick size 10 6 6 point 0 (0,0,3) point 1 (12.7,0,3) ...

point 2 (0,5.5,3) point 3 (0,0,4.8) ratio 0.9 1 0.9

zone create brick size 20 20 20 point 0 (14,0.75,4.8) point 1 (16,0.75,4.8) ...

point 2 (14,4.75,4.8) point 3 (14,0.75,5.71)

zone create brick size 20 6 10 point 0 (0,0,5.71) point 1 (15,0,5.71) ...

point 2 (0,5.5,5.71) point 3 (0,0,10.71) ratio 0.9 1 1.1

zone create brick size 20 6 10 point 0 (15,0,5.71) point 1 (30,0,5.71) ...

point 2 (15,5.5,5.71) point 3 (15,0,10.71) ratio 1.1 1 1.1

zone attach by-face range position-z 3
 zone attach by-face range position-z 4.8
 zone attach by-face range position-z 5.71
 zone attach by-face range position-x 15
 zone attach by-face range position-x 14
 ;assigning groups
 zone group 'Pillar=Anorthosite' range position-z 0 5
 zone group 'Pillar=pegmatoid' range position-z 5 5.2
 zone group 'Pillar=Chromitite' range position-z 5.2 5.6
 zone group 'Pillar=pyroxenite' range position-z 5.6 10.71
 zone face skin slot "Pillar"
 ;Section through pillar
 zone group 'Section=a' range position-y 0 2.75
 zone group 'Section=b' range position-y 2.75 5.5
 zone group 'hist=slice' range position-z 5.175 5.335 ; group used to calculate stress and strain
 ; Assign constitutive models and properties
 zone cmodel assign strain-softening
 zone property density 2749 bulk 67.86e9 shear 9.97e9 range group 'anorthosite'
 zone property cohesion 12.8e6 friction 41.4 tension 2.08e5 dilation 10.4 range group 'anorthosite'
 zone property table-friction 'fri_an' table-cohesion 'coh_an' range group 'anorthosite'
 table 'fri_an' add (0, 36.9) (0.005,36.9) (.02, 36.9) (1, 36.9)
 table 'coh_an' add (0, 12.8e6) (.005, 12.8e6) (.02, 10e6) (1, 0)

zone property density 3968 bulk 26.08e9 shear 10e9 range group 'chromitite'
 zone property cohesion 6.6e6 friction 44.1 tension 0.72e5 dilation 11 range group
 'chromitite'
 zone property table-friction 'fri_cr' table-cohesion 'coh_cr' range group 'chromitite'
 table 'fri_cr' add (0, 35.6) (0.005, 35.6) (.02, 35.6) (1, 35.6)
 table 'coh_cr' add (0, 8.8e6) (0.005, 8.8e6) (.02, 3.4e6) (1, 0)
 zone property density 3224 bulk 23.52e9 shear 15.49e9 range group 'pyroxenite'
 zone property cohesion 10.5e6 friction 42.39 tension 1.48e5 dilation 9.5 range group
 'pyroxenite'
 zone property table-friction 'fri_px' table-cohesion 'coh_px' range group 'pyroxenite'
 table 'fri' add (0, 35) (0.005, 35) (.02, 35) (1, 35)
 table 'coh' add (0, 14.2e6) (0.005, 14.2e6) (.02, 13e6) (1, 0)
 zone property density 2801 bulk 13.14e9 shear 8.27e9 range group 'pegmatoid'
 zone property cohesion 4.9e6 friction 32.8 tension 2.8e5 dilation 8.2 range group
 'pegmatoid'
 zone property table-friction 'fri_pg' table-cohesion 'coh_pg' range group 'pegmatoid'
 table 'fri_pg' add (0, 30) (0.005, 30) (.02, 30) (1, 30)
 table 'coh_pg' add (0, 4.9e6) (0.005, 4.9e6) (.02, 2e6) (1, 8e4)
 ;interface
 zone separate by-face new-side group 'interface1' range group 'pegmatoid' group
 'anorthosite'

zone interface 1 create by-face range group 'Interface1'

zone interface 1 node property stiffness-normal 1e12 stiffness-shear 1e12 ...

friction 30 cohesion 1e6

zone separate by-face new-side group 'interface2' range group 'chromitite' group
'pegmatoid'

zone interface 2 create by-face range group 'Interface2'

zone interface 2 node property stiffness-normal 1e12 stiffness-shear 1e12 ...

friction 33 cohesion 1e6

zone separate by-face new-side group 'interface3' range group 'pyroxenite' group
'chromitite'

zone interface 3 create by-face range group 'Interface3'

zone interface 3 node property stiffness-normal 1e12 stiffness-shear 1e12 ...

friction 35 cohesion 1e6

; Assign Boundary Conditions

zone face apply velocity-normal 0 range position-x 0

zone face apply velocity-normal 0 range position-x 30

zone face apply velocity-normal 0 range position-y 0

zone face apply velocity-normal 0 range position-y 5.5

zone face apply velocity-normal 0 range position-z 0

zone face apply velocity-normal -1e-6 range position-z 10.71

```

fish define setzones

;

minz = 5.175

maxz = 5.335

global slicezones = map

loop foreach local zp zone.list

    if zone.pos.z(zp) > minz & zone.pos.z(zp) < maxz

        slicezones(zone.id(zp)) = zp

    endif

end_loop

;

end

@setzones

fish define load

sum = 0

nzon = 0

loop foreach local zp slicezones

    sum = sum - zone.stress.zz(zp)

    nzon = nzon + 1

end_loop

load = sum / nzon

end

```

```

fish define setzp
;
global gps = map
loop foreach local p zone.list
    if zone.isgroup(p,'slice')
        gps(zone.id(p)) = p
    endif
end_loop
;
end
@setzp

```

```

fish define strain

a = 0

loop foreach local p gps
    a = a - zone.strain.inc.zz(p)

end_loop

strain = a / 1600

end

```

```

; create a group to store the zones in a slice in the middle of the pillar
;zone group 'slice' slot 'hist' range position-z 0.445 0.465

; loop over all zones
; if zone is in this group, add SZZ to a running total
; divide running total by total number of zones in the slice
; load = that average
fish history name 'load' @load
fish history name "strain" @strain

;model step 1000
;model save 'crush pillar'
fish define doplots
;
fname_failure_no_siding = file_root + '_failure_no_siding_' + stagename + '.png'
fname_stress_no_siding = file_root + '_stress_no_siding_' + stagename + '.png'
;
command
;
plot 'failure_no_siding' export bitmap filename [fname_failure_no_siding]
plot 'stress_no_siding' export bitmap filename [fname_stress_no_siding]
;
end_command

```

```

;
end

fish define runmodel

;

totsteps = 700000          ; total number of steps to run analysis

noutputs = 50              ; number of outputs

stepsperstage = int(totsteps/noutputs)

;

loop istage (1,noutputs)

;

  stagename = string(istage)

  command

    model step [stepsperstage]

  end_command

;

  doplots

;

end_loop

;

end

@runmodel

model save 'crush pillar no siding'

```

Data file for call plotview2

plot create 'stress_no_siding'

plot clear

plot active on

plot background 'white'

plot outline active on width 2 color 'black'

plot legend active on heading color 'black' copyright color 'black' ...

placement left size 25,50 ...

step active off ...

time-real active off ...

time-model active off ...

title-customer active off ...

view-info active off

plot title-job active off

plot title active off

plot view projection parallel magnification 0.879886 ...

center (14.659,3.8812,4.4357) eye (14.659,-10.386,4.4357) roll 0.000 ...

clip-front -1e+10 clip-back 1e+10

plot item create chart-history active off ...

history 'load' name "load load (FISH)" style line reversed-x off reversed-y off ...

style-line width 2 style solid color 'seagreen' ...

alias "" ...

vs 'strain' reversed off ...

begin 0 end 0 skip 0 ...

axis-x log off minimum auto maximum auto inside on ...
label "Strain (mm/mm)" exponent show on value -16 auto on ...
axis-y log off minimum auto maximum auto inside on ...
label "Stress (Pa)" exponent show on value -6 auto on ...
title active off ...
labels active on size 20 family 'Times New Roman' style bold ...
marks active on size 18 family 'Times New Roman' style normal ...
chart width 2 box off color 'black' position 1 1 size 96 97 ...
grid active off ...
transparency 0 ...
legend active on ...
title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...
series active on size 44 family 'Arial' style normal color 'black' ...
range off ...
vs active on text "" size 44 family 'Arial' style normal color 'black' ...
vs-range off
plot item create gridpoint-fix active off ...
color-list global on clear ...
label "Fixed in local X" active on color 'cyan' ...
label "Fixed in local Y" active on color 'seagreen' ...
label "Fixed in local Z" active on color 'red' ...
line width 1 style solid ...
scale target 0.02 value 0.317193 automatic ...
map axis xyz translate (0,0,0) scale (1,1,1) ...

def-fac active off ...
 transparency 0 ...
 cut active off ...
 clip active off ...
 legend active on ...
 title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...
 line-count active off ...
 scale active on size 44 family 'Arial' style normal color 'black' ...
 map active on size 44 family 'Arial' style normal color 'black' ...
 def-fac active on size 44 family 'Arial' style normal color 'black'
 plot item create zone active on ...
 contour stress quantity zz stress off log off ...
 method constant null fluid off mechanical off thermal off ...
 ramp rainbow ...
 minimum -3e+08 maximum 0 interval 500000 ...
 reversed off above automatic below automatic ...
 polygons fill on outline active on width 1 color 'black' ...
 outline-transparency 80 lighting on offset 0.5 2 selected-only off selected-highlight
 off ...
 null-faces-only off ...
 map axis xyz translate (0,0,0) scale (1,1,1) ...
 deformation-factor active off ...
 hide-null mechanical on thermal off fluid off ...
 transparency 0 ...

cut active off ...

clip active off ...

legend active on ...

 title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

 map active on size 44 family 'Arial' style normal color 'black' ...

 hide-null active off ...

 deformation-factor active on size 44 family 'Arial' style normal color 'black' ...

 contour labels active on size 44 family 'Arial' style normal color 'black' ...

 labels-maximum 20 ...

 method active on size 44 family 'Arial' style normal color 'black' ...

range group "Section=a" slot "Any" not

plot item create zone active off ...

 contour property log off name 'strain-shear-plastic' ...

 method constant null fluid off mechanical off thermal off ...

ramp rainbow ...

 minimum 0 maximum 0.012 interval 0.0001 ...

 reversed off above automatic below automatic ...

polygons fill on outline active on width 1 color 'black' ...

 outline-transparency 80 lighting on offset 0.5 2 selected-only off selected-highlight off ...

 null-faces-only off ...

map axis xyz translate (0,0,0) scale (1,1,1) ...

deformation-factor active off ...

hide-null mechanical on thermal off fluid off ...

transparency 0 ...

cut active off ...

clip active off ...

legend active on ...

title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

map active on size 44 family 'Arial' style normal color 'black' ...

hide-null active off ...

deformation-factor active on size 44 family 'Arial' style normal color 'black' ...

contour labels active on size 44 family 'Arial' style normal color 'black' ...

labels-maximum 20 ...

method active on size 44 family 'Arial' style normal color 'black'

Data file for call plotview shear2

plot create 'failure_no_siding'

plot clear

plot active on

plot background 'white'

plot outline active on width 2 color 'black'

plot legend active on heading color 'black' copyright color 'black' ...

placement left size 25,50 ...

step active off ...

time-real active off ...

time-model active off ...

title-customer active off ...

view-info active off

plot title-job active off

plot title active off

plot view projection parallel magnification 0.879886 ...

center (14.659,3.8812,4.4357) eye (14.659,-10.386,4.4357) roll 0.000 ...

clip-front -1e+10 clip-back 1e+10

plot item create chart-history active off ...

history 'load' name "load load (FISH)" style line reversed-x off reversed-y off ...

style-line width 2 style solid color 'seagreen' ...

alias "" ...

vs 'strain' reversed off ...

begin 0 end 0 skip 0 ...

axis-x log off minimum auto maximum auto inside on ...

label "Strain (mm/mm)" exponent show on value -16 auto on ...

axis-y log off minimum auto maximum auto inside on ...

label "Stress (Pa)" exponent show on value -6 auto on ...

title active off ...

labels active on size 20 family 'Times New Roman' style bold ...

marks active on size 18 family 'Times New Roman' style normal ...

chart width 2 box off color 'black' position 1 1 size 96 97 ...

grid active off ...

transparency 0 ...

legend active on ...

title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...

series active on size 44 family 'Arial' style normal color 'black' ...
 range off ...
 vs active on text "" size 44 family 'Arial' style normal color 'black' ...
 vs-range off
 plot item create gridpoint-fix active off ...
 color-list global on clear ...
 label "Fixed in local X" active on color 'cyan' ...
 label "Fixed in local Y" active on color 'seagreen' ...
 label "Fixed in local Z" active on color 'red' ...
 line width 1 style solid ...
 scale target 0.02 value 0.317193 automatic ...
 map axis xyz translate (0,0,0) scale (1,1,1) ...
 def-fac active off ...
 transparency 0 ...
 cut active off ...
 clip active off ...
 legend active on ...
 title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...
 line-count active off ...
 scale active on size 44 family 'Arial' style normal color 'black' ...
 map active on size 44 family 'Arial' style normal color 'black' ...
 def-fac active on size 44 family 'Arial' style normal color 'black'
 plot item create zone active on ...
 contour property log off name 'strain-shear-plastic' ...

method constant null fluid off mechanical off thermal off ...
 ramp rainbow ...
 minimum 0 maximum 0.012 interval 0.0001 ...
 reversed off above automatic below automatic ...
 polygons fill on outline active on width 1 color 'black' ...
 outline-transparency 80 lighting on offset 0.5 2 selected-only off selected-highlight
 off ...
 null-faces-only off ...
 map axis xyz translate (0,0,0) scale (1,1,1) ...
 deformation-factor active off ...
 hide-null mechanical on thermal off fluid off ...
 transparency 0 ...
 cut active off ...
 clip active off ...
 legend active on ...
 title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...
 map active on size 44 family 'Arial' style normal color 'black' ...
 hide-null active off ...
 deformation-factor active on size 44 family 'Arial' style normal color 'black' ...
 contour labels active on size 44 family 'Arial' style normal color 'black' ...
 labels-maximum 20 ...
 method active on size 44 family 'Arial' style normal color 'black' ...
 range group "Section=a" slot "Any" not
 plot item create zone active off ...

contour property log off name 'strain-shear-plastic' ...
method constant null fluid off mechanical off thermal off ...
ramp rainbow ...
minimum 0 maximum 0.012 interval 0.0001 ...
reversed off above automatic below automatic ...
polygons fill on outline active on width 1 color 'black' ...
outline-transparency 80 lighting on offset 0.5 2 selected-only off selected-highlight
off ...
null-faces-only off ...
map axis xyz translate (0,0,0) scale (1,1,1) ...
deformation-factor active off ...
hide-null mechanical on thermal off fluid off ...
transparency 0 ...
cut active off ...
clip active off ...
legend active on ...
title active on text "" size 55 family 'Times New Roman' style bold color 'black' ...
map active on size 44 family 'Arial' style normal color 'black' ...
hide-null active off ...
deformation-factor active on size 44 family 'Arial' style normal color 'black' ...
contour labels active on size 44 family 'Arial' style normal color 'black' ...
labels-maximum 20 ...
method active on size 44 family 'Arial' style normal color 'black'

Worst-case scenario

Model new

;Grid generation

zone create brick size 20 20 80 point 0 (0,0,0) point 1 (100,0,0) ...

point 2 (0,32,0) point 3 (0,0,40.91) ratio 0.88 1 1

zone create brick size 10 10 0 point 0 (100,0,0) point 1 (114,0,0) ...

point 2 (100,32,0) point 3 (100,0,18.2) ratio 1 1 0.9

zone create brick size 6 10 10 point 0 (100,0,18.2) point 1 (108.35,0,18.2) ...

point 2 (100,32,18.2) point 3 (100,0,20)

zone create brick size 3 10 10 point 0 (109.65,0,18.2) point 1 (114,0,18.2) ...

point 2 (109.65,32,18.2) point 3 (109.65,0,20)

zone create brick size 10 10 40 point 0 (100,0,20.91) point 1 (114,0,20.91) ...

point 2 (100,32,20.91) point 3 (100,0,40.91) ratio 1 1 1.1

zone create brick size 40 40 40 point 0 (100,0,20) point 1 (104,0,20) ...

point 2 (100,1,20) point 3 (100,0,20.91)

zone create brick size 40 40 40 point 0 (100,31,20) point 1 (104,31,20) ...

point 2 (100,32,20) point 3 (100,31,20.91)

zone create brick size 20 20 80 point 0 (114,0,0) point 1 (214,0,0) ...

point 2 (114,32,0) point 3 (114,0,40.91) ratio 1.12 1 1

zone attach by-face range position-x 114

zone attach by-face range position-x 100

zone attach by-face range position-z 20

zone attach by-face range position-z 20.91

zone attach by-face range position-z 18.2

zone group 'hist=slice' range position-z 20.355 20.555 ; group used to calculate stress and strain

zone group 'hist=slice1' range position-z 37.91 40.91

;Assign Boundary Conditions

zone face apply velocity-normal 0 range position-x 0

zone face apply velocity-normal 0 range position-x 214

zone face apply velocity-normal 0 range position-y 0

zone face apply velocity-normal 0 range position-y 32

zone face apply velocity-normal 0 range position-z 0

zone face apply velocity-normal -1e-6 range position-z 40.91

;assigning groups

zone group 'Pillar=Anorthosite' range position-z 0 20.2

zone group 'Pillar=pegmatoid' range position-z 20.2 20.4

zone group 'Pillar=Chromitite' range position-z 20.4 20.8

zone group 'Pillar=pyroxenite' range position-z 20.8 40.91

; Assign constitutive models and properties

zone cmodel assign elastic

zone property density 2749 bulk 67.86e9 shear 9.97e9 range group 'anorthosite'

zone property density 3968 bulk 26.08e9 shear 10e9 range group 'chromitite'

zone property density 3224 bulk 23.52e9 shear 15.49e9 range group 'pyroxenite'

zone property density 2801 bulk 13.14e9 shear 8.27e9 range group 'pegmatoid'

fish define setzones

;

minz = 20.355

```

maxz = 20.555

global slicezones = map

loop foreach local zp zone.list

    if zone.pos.z(zp) > minz & zone.pos.z(zp) < maxz

        slicezones(zone.id(zp)) = zp

    endif

end_loop

;

end

@setzones

fish define load

    sum = 0

    nzon = 0

    loop foreach local zp slicezones

        sum = sum - zone.stress.zz(zp)

        nzon = nzon + 1

    end_loop

    load = sum / nzon

end

fish define setzp

;

global gps = map

loop foreach local p zone.list

    if zone.isgroup(p,'slice')

```

```

        gps(zone.id(p)) = p
    endif
end_loop
;
end
@setzp
fish define strain
a = 0
loop foreach local p gps
a = a - zone.strain.inc.zz(p)
end_loop
strain = a / 25600
end
fish define setzones1
;
minz = 37.91
maxz = 40.91
global slicezones1 = map
loop foreach local zp zone.list
    if zone.pos.z(zp) > minz & zone.pos.z(zp) < maxz
        slicezones1(zone.id(zp)) = zp
    endif
end_loop
;

```

```

end

@setzones1

fish define Stressmodel

    sum = 0

    nzon = 0

    loop foreach local zp slicezones1

        sum = sum - zone.stress.zz(zp)

        nzon = nzon + 1

    end_loop

    Stressmodel = sum / nzon

end

; Histories

history interval 1

fish history name 'load' @load

fish history name 'stressmodel' @Stressmodel

fish history name "strain" @strain

model step 20000

model save 'worst case correct'

```

Best-case scenario

```

Model new

;Grid generation

zone create brick size 3 2 80 point 0 (-41,0,0) point 1 (100,0,0) ...
                                point 2 (-41,32,0) point 3 (-41,0,40.91) ratio 0.88 1 1

```

zone create brick size 10 10 20 point 0 (100,0,0) point 1 (114,0,0) ...
 point 2 (100,32,0) point 3 (100,0,18.2) ratio 1 1 0.9
 zone create brick size 10 2 20 point 0 (100,0,18.2) point 1 (108.35,0,18.2) ...
 point 2 (100,32,18.2) point 3 (100,0,20)
 zone create brick size 3 5 20 point 0 (109.65,0,18.2) point 1 (114,0,18.2) ...
 point 2 (109.65,32,18.2) point 3 (109.65,0,20)
 zone create brick size 5 5 20 point 0 (100,0,20.91) point 1 (114,0,20.91) ...
 point 2 (100,32,20.91) point 3 (100,0,40.91) ratio 1 1 1.1
 zone create brick size 40 40 40 point 0 (100,0,20) point 1 (104,0,20) ...
 point 2 (100,1,20) point 3 (100,0,20.91)
 zone create brick size 40 40 40 point 0 (100,31,20) point 1 (104,31,20) ...
 point 2 (100,32,20) point 3 (100,31,20.91)
 zone create brick size 3 2 80 point 0 (114,0,0) point 1 (255,0,0) ...
 point 2 (114,32,0) point 3 (114,0,40.91) ratio 1.12 1 1
 zone create brick size 3 1 20 point 0 (-191,0,0) point 1 (-41,0,0) ...
 point 2 (-191,32,0) point 3 (-191,0,20)
 zone create brick size 3 1 20 point 0 (255,0,0) point 1 (405,0,0) ...
 point 2 (255,32,0) point 3 (255,0,20)
 zone create brick size 3 1 20 point 0 (255,0,20.91) point 1 (405,0,20.91) ...
 point 2 (255,32,20.91) point 3 (255,0,40.91)
 zone create brick size 3 1 20 point 0 (-191,0,20.91) point 1 (-41,0,20.91) ...
 point 2 (-191,32,20.91) point 3 (-191,0,40.91)
 zone attach by-face range position-x -41
 zone attach by-face range position-x 255

zone attach by-face range position-x 114
 zone attach by-face range position-x 100
 zone attach by-face range position-z 20
 zone attach by-face range position-z 20.91
 zone attach by-face range position-z 18.2
 zone group 'hist=slice' range position-z 20.355 20.555 ; group used to calculate stress
 and strain
 zone group 'hist=slice1' range position-z 37.91 40.91
 ;Assign Boundary Conditions
 zone face apply velocity-normal 0 range position-x -191
 zone face apply velocity-normal 0 range position-x 405
 zone face apply velocity-normal 0 range position-y 0
 zone face apply velocity-normal 0 range position-y 32
 zone face apply velocity-normal 0 range position-z 0
 zone face apply velocity-normal -1e-6 range position-z 40.91
 ;assigning groups
 zone group 'Pillar=Anorthosite' range position-z 0 20.2
 zone group 'Pillar=pegmatoid' range position-z 20.2 20.4
 zone group 'Pillar=Chromitite' range position-z 20.4 20.8
 zone group 'Pillar=pyroxenite' range position-z 20.8 40.91
 ; Assign constitutive models and properties
 zone cmodel assign elastic
 zone property density 2749 bulk 67.86e9 shear 9.97e9 range group 'anorthosite'
 zone property density 3968 bulk 26.08e9 shear 10e9 range group 'chromitite'

```

zone property density 3224 bulk 23.52e9 shear 15.49e9 range group 'pyroxenite'
zone property density 2801 bulk 13.14e9 shear 8.27e9 range group 'pegmatoid'
fish define setzones
;
minz = 20.355
maxz = 20.555
global slicezones = map
loop foreach local zp zone.list
    if zone.pos.z(zp) > minz & zone.pos.z(zp) < maxz
        slicezones(zone.id(zp)) = zp
    endif
end_loop
;
end
@setzones
fish define load
sum = 0
nzon = 0
loop foreach local zp slicezones
    sum = sum - zone.stress.zz(zp)
    nzon = nzon + 1
end_loop
load = sum / nzon
end

```

```

fish define setzp
;
global gps = map
loop foreach local p zone.list
    if zone.isgroup(p,'slice')
        gps(zone.id(p)) = p
    endif
end_loop
;
end

@setzp

fish define strain
a = 0
loop foreach local p gps
a = a - zone.strain.inc.zz(p)
end_loop
strain = a / 25600
end

fish define setzones1
;
minz = 37.91
maxz = 40.91
global slicezones1 = map
loop foreach local zp zone.list

```

```

    if zone.pos.z(zp) > minz & zone.pos.z(zp) < maxz
        slicezones1(zone.id(zp)) = zp
    endif
end_loop
;
end

@setzones1

fish define Stressmodel

    sum = 0

    nzon = 0

    loop foreach local zp slicezones1
        sum = sum - zone.stress.zz(zp)
        nzon = nzon + 1
    end_loop

    Stressmodel = sum / nzon

end

; Histories

history interval 1

fish history name 'load' @load

fish history name 'stressmodel' @Stressmodel

fish history name "strain" @strain

model step 20000

model save 'best case'

```