Evaluation of the performance of shotcrete with and without fibre reinforcement under dynamic and quasi-static loading conditions.

Volume I


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Executive summary

Introduction
The primary output of the project is the quantification of in situ performance of shotcrete under high stress, dynamic loading and deadweight conditions.

Methodology
A literature survey and a survey of the use of shotcrete in the South African mining industry were conducted in the early stages of the project.

Several underground sites were selected for underground monitoring of shotcrete, which cover a range of rock mass and stress conditions. These have been instrumented and monitored over the life of the project. At some of sites large displacements were experienced which caused significant damage to the shotcrete. This enabled the investigation of the interaction of shotcrete with the rock mass.

Non-linear modelling of various rock masses under different stress levels was conducted to quantify the effects of quasi-static loading. A series of ground reaction charts has been developed which can be used to estimate the maximum expected displacement.

A programme of laboratory tests was conducted, which included standard beam and panel tests, UCS tests and measurement of the fibre content.

Yield line analysis was used to quantify the performance of shotcrete. This was applied to the standard test panels and the required shotcrete characteristics have been determined for design. It was also used to estimate the shotcrete capacity at the underground monitoring sites.

A methodology for the design of shotcrete has been determined for application in underground mines. The method is based on the underground monitoring, numerical modelling, laboratory testing and yield line analysis. It summarises the important shotcrete characteristics and rock engineering inputs required for the design of shotcrete. Design charts are included to simplify the methodology.
Summary of results and conclusions

Underground monitoring

Five test sites were identified, established and instrumented at three different South African mines.

- At the two South Deep Mine sites quasi-static pillar loading led to high deformations of 70 mm and more over the 14 months of testing. The South Deep sites were ideal for the analysis of shotcrete failure mechanisms. The sites were also exposed to strong ground motions from both seismicity and nearby bench blasts, further adding to the contributions they have made.

- The Mponeng Mine 109 site captures the performance of fibre reinforced shotcrete at deep level mining depths where over-stopping or de-stressing conditions prevail.

- The Mponeng Mine 116 test site was established to investigate the influence of strong ground motions from seismicity on the performance of shotcrete.

- The Impala Platinum Mine site captures the effectiveness of un-reinforced shotcrete at controlling spalling ground conditions at intermediate mining depths where moderate stress changes are expected.

Thorough site assessments have led to the following conclusions and improved understanding.

- At Mponeng Mine 116 level induced deformation is related to the calculated PPV of the seismic event responsible for the strong ground motion.

- Instantaneous jumps in sidewall deformation were measured at the South Deep sites at the time of nearby longhole blast events. The vibrational intensity of a blast event has been found to affect the degree of deformation that results.

- In depth analysis of shotcrete crack formation and propagation at the South Deep sites identified that shotcrete fails in two distinct stages.
  - The primary stage of failure is identified by the formation and propagation of individual “primary” cracks throughout the applied shotcrete. This stage of failure is not believed to result in a noticeable drop in the performance of installed shotcrete.
  - The secondary stage follows once primary cracks have propagated far enough to join or interact. In many cases the joining of primary cracks is achieved through
the development of secondary cracking. This stage of shotcrete failure is synonymous with a marked drop in the performance of the applied shotcrete and is accompanied by significant increases in sidewall deformation and damage.

- It is suggested that primary cracks form mainly in flexure. The damage caused by these cracks and its influence on the overall shotcrete performance is localised and thus rather limited.
- Cracking during the secondary stage of shotcrete failure has been observed to commonly occur in flexure. Secondary cracking has a substantial influence on the overall performance of shotcrete.

**Non-linear modelling**

- A generic model has been created with the discrete element code UDEC, using a voronoi tessellation scheme to permit examination of the effect of rock fragmentation on support requirements.
- The model has been used to examine the relationship between support pressure and rock wall deformation in terms of Ground Reaction Curves (GRC) for a range of common rock mass circumstances and stress regimes that occur in both the gold and platinum/chrome tabular mines. A summary of maximum sidewall deformations is presented. These are related to rock type, rock mass rating in the form of GSI, and an estimate of the maximum stress concentration induced in the tunnel wall. The listed values are derived from models of 3.5 x 3.5 m tunnels. For other excavation sizes, a scaling factor can be applied.
- For selection of appropriate shotcrete systems for a tunnel in a certain rock mass under an estimated stress field the Ground Reaction Curves can be used to supply estimates of required support pressures to limit deformations, plus maximum movements in the tunnel wall. Considerations and modifications to the basic GRCs include scaling for excavation size, potential for extreme squeezing (deformations that are large and relatively indeterminate), and whether the tunnel will be subjected to stress change, including de-stressing.
- With excavation support requirements defined in terms of deflection, estimates of required shotcrete strength, thickness and deformability would be made using yield line theory.
Laboratory testing

- The laboratory test results have revealed that the incorporation of fibres in shotcrete has little effect on the modulus of rupture and the uniaxial compressive strength of shotcrete. However, the incorporation of fibres improves the post peak performance of shotcrete by offering it residual strength and energy absorption capacities.

- The shape of the specimens has an effect on the performance of shotcrete. This is especially the case with EFNARC panels due to the nature of the testing method, which requires the panel base to be flush with the stand on which the panel is placed during testing.

- It has been noted that the performance of fibre reinforced shotcrete depends on the types of fibres used in the shotcrete.

Yield line analysis

- A yield line solution for shotcrete and bolts have been developed.

- The demand and capacity of shotcrete support can be estimated using yield line analysis.

- The displacements calculated from numerical modelling can be used as a preliminary estimate of quasi-static loading demand, where measurements are not available. It is assumed that the shotcrete will deform by that amount and may enter its residual state, limiting its remaining capacity.

- The rock load demand from deadweight loading is estimated using a roof prism formed between tendons. It is relatively low for the tendon spacing (1.0 m to 1.5 m) used in the underground sites. However, the rock load demand increases with the cube of the spacing and this will become a concern for greater tendon spacing.

- The dynamic loading demand can be estimated by assessing the kinetic and potential energy of a prism of rock formed between tendons. The demand increases with the cube of the spacing and the square of the ejection velocity.

- The deflections at peak load ($\delta_p$) in both EFNARC and ASTM C1550/RDP tests are highly variable, but can be used as an indication of whether, the peak load is likely to be exceeded. The $\delta_p$ values for unreinforced and polypropylene fibre reinforced shotcrete are much lower than for steel fibre reinforced shotcrete.
- The peak moment capacity of steel fibre reinforced shotcrete can be determined reliably using ASTM C1550/RDP test results.

- The peak loads and moduli in EFNARC steel fibre reinforced tests are quite variable and this is probably due to the non-unique crack pattern. The moment capacities are however, similar to the ASTM C1550/RDP test results.

- Unreinforced shotcrete and polypropylene fibre reinforced shotcrete have highly variable peak moment capacities.

- Peak moment capacity is proportional to steel fibre density or mesh area and increases with the cube of the shotcrete thickness.

- The residual moment capacities can be reliably determined for both steel and polypropylene fibre reinforced shotcrete. The residual strength increases with increasing fibre content.

- Steel fibre reinforced shotcrete will control the displacement rate more effectively than polypropylene fibre reinforced shotcrete. Polypropylene fibre reinforced shotcrete can accommodate larger deformations.

- Unreinforced shotcrete does not have residual strength and therefore should not be used where quasi-static deformation or dynamic loading is expected.

- Energy absorption increases with increasing fibre content.

- The shotcrete capacity increases with increasing thickness.

- Steel fibre reinforced shotcrete has higher Energy absorption capacity prior to any deformation taking place, but after some initial deformation has taken place, the remaining energy absorption capacity of polypropylene fibre reinforced shotcrete is higher.

- The potential corrosion of steel fibre after cracking should also be considered.

- The large scale panel tests indicate that the capacity of shotcrete does not increase quite as expected with increasing shotcrete thickness and for uniformly distributed loading. This must be taken into account in the factor of safety for design. More work is required to verify the actual relations.

- Actual underground capacities are highly variable and dependant on the local rock surface geometry and shotcrete application.
Shotcrete design methodology

- The design methodology developed can be readily applied to shotcrete design for underground excavations and caters for a variety of rock mass and loading conditions.
- It specifically assesses the quasi-static, deadweight and dynamic loading conditions.
- The required inputs for the design of shotcrete are listed and discussed.
- A required testing programme for determining shotcrete characteristics is described.
- The assessment of demand for each loading condition is described and design charts and tables are provided for each condition.
- The shotcrete capacity is determined from laboratory testing of shotcrete panels. The ASTM C1550 round panel test is recommended for design.
- Factors of safety are recommended for each loading condition, which take the uncertainty into consideration.
- Recommendations for monitoring of shotcrete performance are provided.

The following is recommended for further research:

- There is scope for further study into a relationship between PPV and induced instantaneous deformation. The existence of a “no deformation” threshold needs to be confirmed and the relationship after this threshold needs to be quantified.
- There is scope to further study the depth of fracturing in deep level, high deformation environments. Large variations have been noted to exist between depths of fracturing as measured using different types of instruments. It is believed that the intensity of fracturing is probably locally variable and the behaviour is more complex than may be expected.
- The concept of the primary and secondary stages of shotcrete failure deserves further testing. If possible, back analysis of underground failure incidents should be conducted assessing the stage and degree of shotcrete failure at the time of site collapse.
  - There is scope to develop a surface test under controlled laboratory conditions that can effectively demonstrate and repeatedly produce a progression from primary to secondary stage failure as defined in this research.
If such a test can be devised then it would be useful to conduct a series of tests that investigate the effect of varying common design parameters like strength, thickness and type of reinforcement on the duration of the primary stage of failure as identified by this work. Can the shotcrete be engineered to effectively prolong the time before entering the secondary stage?

- It is recommended that a further programme of large scale panel tests should be carried out, with varying tendon spacing, shotcrete thickness, different types and quantities of reinforcement for uniformly distributed and point loads. The data should be analysed with yield line theory.

- A small round panel test method should be developed for quality control purposes. This will be easier to transport underground.

- A dynamic test programme should be developed with a basic test configuration that can easily be quantified. This should also include panel tests with varying tendon spacing, shotcrete thickness and different types and quantities of reinforcement.
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1 Introduction

The contribution made by shotcrete to the support of mine tunnels, and a methodology for determining the shotcrete requirements in terms of strength, thickness and flexibility has long been a debate in the rock engineering community.

This project was initiated largely due to investigations of rockburst incidents that resulted in the following question being posed “Do we know and understand what we are designing for?” This is especially relevant to the use of shotcrete in highly stressed tunnels that may be subject to dynamic loading.

The project primarily focuses on the underground monitoring of shotcrete under quasi-static and potentially dynamic loading conditions. The RETAC committee requested that the project should also include deadweight loading. It was agreed that the deadweight loading would be investigated through a literature survey only.

The primary output of the project is the “Quantification of in situ performance of shotcrete under high stress, dynamic loading and deadweight conditions.” The following outputs were stipulated in the proposal:

- The relationships between common laboratory tests (UCS and Energy absorption etc) and the actual performance underground (i.e. What does a UCS of 40 MPa and an EA of 700J mean underground for a 25mm, 50mm, 100mm etc. thickness of shotcrete under different loading conditions) will be determined and calibrated.

- The interaction of the shotcrete with the rock mass will be determined.

- A methodology for shotcrete design in the form of design charts.

- Description of shotcrete materials, equipment and application by the mine.

- Testing and design of fibre reinforced shotcrete.

It was proposed that the technology used in applying the shotcrete and the results of the study will be transferred to the industry as follows:

- through presentations to the mining and rock engineering personnel on the mine,
- through presentation at workshops of the interim results obtained during the study
- by compiling a report on the findings of the study that will be available through SIMRAC,
by providing the rock engineering practitioner with a methodology for shotcrete design in the form of design charts and handbook,

by publishing a paper on the findings of the study.

A literature survey and a survey of the use of shotcrete in the industry were conducted in the early stages of the project.

One of the main objectives of this research project is to investigate and understand the interaction of shotcrete and rock under quasi-static and dynamic conditions. A number of underground sites were selected to cover a range of rock mass and stress conditions. These have been instrumented and monitored over the life of the project. At some sites large displacements were experienced which caused significant damage to the shotcrete. This enabled the investigation of the interaction of shotcrete with the rock mass.

Non-linear modelling of various rock masses under different stress levels was conducted to quantify the effects of quasi-static loading. A series of ground reaction charts has been developed which can be used to estimate the maximum expected displacement.

A programme of laboratory tests was conducted, which included standard beam and panel tests, UCS tests and measurement of the fibre content.

Yield line analysis was used to quantify the performance of shotcrete support. This was applied to the standard test panels and the required shotcrete characteristics have been determined for design. It was also used to estimate the shotcrete capacity at the underground monitoring sites.

A methodology for the design of shotcrete support has been determined for application in underground mines. The method is based on the underground monitoring, numerical modelling, laboratory testing and yield line analysis. It summarises the important shotcrete characteristics and other rock engineering inputs required for the design of shotcrete support. Design charts are included to simplify the methodology. This provides a practical tool for use by rock engineering practitioners in designing shotcrete support in underground excavations.

Conclusions and recommendations for further research are presented at the end of the report.
2 Literature review on shotcrete design

2.1 The Function of Shotcrete

Broadly speaking, the purpose of shotcrete is to support rock or soil during and after construction of excavations such as tunnels, slopes, shafts etc. From a design perspective however, one needs to be more specific in terms of the support contribution attributed to shotcrete. The following sections describe the various functions of shotcrete as found in literature.

2.1.1 Supporting the Rock Mass

Originally in Civil Engineering tunnelling, tunnel support was designed to carry the entire load of ground above the tunnel. The assumption was made that the soil or rock has no strength of its own and had to be fully supported by the tunnel support system. This assumption led to the design of thick, stiff and uneconomical tunnel linings that did not perform well under dynamic loading or in squeezing ground conditions. The reason for this behaviour is that the stresses imposed on a stiff support system by dynamic loading or squeezing ground often exceed the strength of any stiff support system and the key to successful rock support under these conditions is support ductility while retaining the ability to apply support pressure.

2.1.2 Support Pressure

It has been proven that even soft rocks or soils have some capacity to carry their own load, provided that some support pressure is applied. Intuitively this property has been used since ancient times in underground excavations such as mine tunnels and Khanats (irrigation adits used by several ancient civilizations). The function of the shotcrete and other support changed from applying support pressure to enhancing the load carrying abilities of the in-situ ground.

The familiar ground reaction and support reaction curves (Figure 2-1) resulted from the new philosophy. The ground and support reaction curves imply that if radial deformation of the tunnel behind the face is allowed, the required load on the shotcrete (support) to stabilize the tunnel would be decreased, allowing for a more cost effective design.
Figure 2-1: Conceptual Support Reaction Curve (Brown, 1999)

The principles of the support pressure design theory are as follows:

- Support should be installed at the correct moment after excavation to allow the rock mass to accept some load before being supported. If support is installed too early, the amount of support required is excessive and uneconomical. If support is installed too late, the excavation would collapse before the support could carry out its intended function;

- Support of the correct stiffness should be installed. If the installed support is not stiff enough, the support reaction and ground reaction curves never intersect and equilibrium is not attained, resulting in tunnel collapse. If the support installed is too stiff, the support
reaction and ground reaction curves would intersect at low deformation and the support would be forced to accept high load. The resulting consequence is that the support will yield prematurely or that the support design is more expensive than required; and

- If support of the correct stiffness is installed at the right time, the ground reaction and support reaction curves intersect near the optimal position and the tunnel is safely supported with optimal shotcrete dimensions.

2.1.3 Holding up Key Blocks

Often in hard rock mines without squeezing ground conditions, tunnels are self supporting except for kinematically feasible rock blocks that may loosen and fall, or for ravelling that may take place over a period of time. The key block theory (Goodman and Shi, 1985) states that if all potentially unstable blocks surrounding a tunnel are supported, then, stability of the tunnel is achieved. The purpose of shotcrete in these conditions would be simply to prevent rock blocks from failing into the tunnel. If bolt or anchor support is installed, the shotcrete’s only function may be to support rock blocks existing between bolt support and to prevent unraveling. In these cases shotcrete design depends on the bolt spacing.

2.1.4 Shotcrete as Load Path Conduit

Shotcrete can act as a conduit to channel load imposed on it by rock or soil to the rest of the support system. Examples of this application are reinforced shotcrete arches between soldier piles in deep basement excavations, shotcrete panels between rock bolts and shotcrete arches used as tunnel support.

2.1.5 Indirect Shotcrete Support

Apart from the direct functions of shotcrete as mentioned above, shotcrete also carries out the following unquantifiable stabilising functions according to Golser (1976), Potvin et al (2005) and Stacey et. al. (2009):

- Shotcrete prevents initial movement along joint planes from developing, thereby stabilising the tunnel surface. The shotcrete also rounds off the sharp corners of a tunnel thereby eliminating stress concentrations which results in failure;
- Shotcrete acts as a strengthening outer layer to the rock. Due to adhesion the rock and shotcrete acts as a unit with enhanced strength;
- The shotcrete retards or prevents the rock from weathering, thereby preventing rock strength reduction (insulation from moisture, air and running water); and
- The shotcrete prevents the additional loosening of the rock mass.
- Shotcrete can penetrate into joints and cracks to produce a wedging effect like mortar in a wall or arch.

2.1.6 Conclusion

Shotcrete can be used as tunnel support by applying support pressure to the rock mass, enhancing rock mass strength, and stabilising key blocks or to retain the entire weight of a failing rock mass. In order to successfully design shotcrete, the specific function of using shotcrete should be identified in order to apply the correct design procedure. The indirect support effects provided by shotcrete can occur simultaneously to provide a more effective combined stabilising effect to the rock surface.

2.2 Shotcrete Failure Modes

Barrett and McCreath (1995) described six basic modes for failure for shotcrete (Figure 2-2). These are:
- Adhesive failure,
- Direct Shear failure,
- Compressive failure,
- Flexural failure,
- Punching shear failure and
- Tensile failure

Descriptions of the modes of failure are summarised from Barrett and McCreath (1995) in the following subsections.
2.2.1 Adhesive failure

Loss of the adhesive bond between the shotcrete and the rock is a common problem if the rock surface is not well prepared i.e. there is mud, dirt or oil, or because the rock itself is weak in tension (highly foliated or closely bedded). Fernandez-Delgado et al (1976) attributed adhesive failure to the thickness of the shotcrete, with failures more prevalent when shotcrete thickness is more than approximately 30mm to 50mm. Whilst this may be true for the conditions in which the tests were done, this assumption is narrow, as this failure may not be observed where the rock surface has been well prepared before applying the shotcrete.

2.2.2 Direct shear failure

This normally takes place when the shotcrete-rock bond is strong enough to resist adhesion loss. Failure of the shotcrete then occurs in direct shear when the applied load exceeds the
shear strength of the shotcrete. The failure develops along the perimeter of the base of the wedge or block in planes parallel to the direction of shear.

2.2.3 Flexural failure

If the shotcrete de-bonds (loses adhesion), the shotcrete can be effective in preventing the rock mass from loosening by acting as a slab in bending, provided it is suitably reinforced and integrated with a system of pattern bolts.

Observations have shown that adhesive failure precedes flexural failure, and both mechanisms are commonly seen together. In fact, the flexural failure mechanism is only kinematically possible if adhesion is lost as a result of peeling off the shotcrete from the rock or slabbing of the substrate.

Tensile fractures will develop on the outer surface of shotcrete in the centre of the slab, where the tensile stress is greatest.

2.2.4 Punching shear failure

This takes place close to the supports for de-bonded shotcrete where the shear forces are at a maximum. Failure does not occur along a plane normal to the shotcrete rock interface, but along planes aligned at approximately 45° to the shotcrete rock interface, perpendicular to the diagonal tensile stresses in the slab. The shotcrete actually fails in tension rather than in shear, but it is the shear load that induces diagonal tensile failure. This failure mode is termed diagonal tensile failure by Vandewalle (1997) and Fernandez-Delgado (1976).

2.2.5 Compressive and tensile failure

Shotcrete may also fail in tension or in compression. This occurs when the induced tensile or compressive stresses in the shotcrete, caused by stress changes in the rock result in fracturing or spalling of the shotcrete.

2.2.6 Conclusions on failure modes

Barrett and McCreath (1995) developed deterministic models for the adhesive, direct shear, flexural and punch shear modes of failure. Analysis of these modes, leads to the conclusion that loss of adhesion significantly reduces the capacity of shotcrete to support the applied loads, by the changing the controlling failure mode from direct shear to flexural failure. The direct shear strength of shotcrete is significantly greater than the flexural strength. However, Fernandez-Delgado (1976) found that in all his large scale tests where the shotcrete lining was
thinner than 50 mm, the shotcrete linings failed through the material rather than at the bond. The shotcrete thickness in Barrett and McCreath’s analyses was 75 mm and the shotcrete properties were different. This illustrates the importance of analysing all failure modes for a given set of shotcrete properties.

Morton et. al (1998) found that the failure mechanism was a combination of adhesion and flexural failure during their large punch testing of shotcrete (35 mm, 60 mm and 80 mm thick) applied to substrates comprising thin square sandstone panels. The composite sandstone shotcrete panels were fixed at the edges (clamped into a frame) and the punch load was applied through a 500 mm disk cut into the centre of the sandstone panel, prior to spraying. Initial cracking occurred prior to measurable adhesion failure (deflection of the substrate panel was also measured and recovery was observed when adhesion loss occurred and the net deflection of the substrate was zero after adhesion loss was complete). In these tests, flexural cracks could develop prior to adhesion failure, since the substrate panel was also bending. Barrett and McCreath suggest that care must be exercised when estimating shotcrete capacity in cases where the adhesive bond is expected to be poor, or in cases where planes of weakness (ie bedding foliation or stress induced slabbing) run sub-parallel to the excavation profile.

The results also show that punching shear strength is greater than the flexural strength, but less than direct shear strength. The models assume a uniformly distributed load applied to a shotcrete panel. If there are point loads close to the supports, the punch shear mode of failure can occur. In practice, the flexural and punch shear modes of failure are most commonly observed. Modes of failure were analysed during the underground monitoring programme (section 4.4.6).

### 2.3 Shotcrete Failure Mechanisms

The following sub-sections discuss the various mechanisms through which excavations apply load to shotcrete and how the shotcrete fails. These mechanisms were explored further during the underground monitoring (section 4.5).

#### 2.3.1 Keyblock loading

Kinematically feasible rock blocks driven by gravity, water pressure in discontinuities and/or rock stress will apply a load on the shotcrete. Stacey (2001) describes how rock loading can affect the behaviour of shotcrete. If shotcrete is applied as a continuous or semi-continuous arch, the
rock applies a distributed load to the shotcrete. This load is usually not uniform and leads to (a) bending moments and (b) shear forces in the shotcrete (Figure 2-3). The direction and magnitude of this loading depend on the stress regime in the rock around an excavation, the shape of the excavation and the dimensions of the shotcrete.

![Diagram](image)

(a) Bending failure
(b) Shear failure

**Figure 2-3:** Bending (flexural) and Shear failures of shotcrete (Stacey, 2001)

In hard rock excavations, preventing such blocks from falling is often the only purpose of using shotcrete, and shotcrete should be designed to withstand these loads.

### 2.3.2 Squeezing Ground

Squeezing ground is found in excavations in ductile rock where the stress is high enough to cause large permanent deformations into the excavation. Barton *et al.* (1974) have defined squeezing rock as ‘plastic flow of incompetent rock under the influence of high pressure.’ Potvin and Hadjigeorgiou (2008) describe three mechanisms of squeezing (Figure 2-4).

![Diagram](image)

a) Complete shear failure
b) Buckling failure
c) Tensile splitting shearing and sliding

**Figure 2-4:** Failure mechanisms for tunnels in squeezing ground (after Potvin and Hadjigeorgiou, 2008)
These deformations are rarely uniform across the shotcrete lining and cause bending moments and shear forces that can be uneconomical to support in mining situations. Often squeezing ground is associated with weak layers in more competent layered rocks (Figure 2-4c). In these cases, the squeezing ground applies localised punch loads to the shotcrete instead of distributed loads. Bosman et. al (2000) describe the occurrence of this type of squeezing ground at Hartebeestfontein mine. Under these conditions, the support is designed to control rather prevent deformation.

2.3.3 Bulking of Brittle Stressed Rock

In excavations with a high stress to strength ratio in brittle rock, the rock mass around an excavation often fractures. The fractured rock increases in volume and will spall into the excavation if not supported.

A uniformly distributed load is usually applied to the excavation. Even if there is good adhesion between the shotcrete and rock, the fractures allow flexural failure (Figure 2-5) or punch shear failure to take place. These loads can be excessive and deformations can be very large, and it may be necessary to design to control rather than prevent deformation under these conditions.

![Buckling (flexural) failure of shotcrete (Stacey, 2001).](image)

**Figure 2-5:** Buckling (flexural) failure of shotcrete (Stacey, 2001).

2.3.4 Seismic Loading

Dynamic loading in mining is experienced when mine-induced seismicity causes rockbursts in which the rock support elements are dynamically loaded. The violent ejection of rock and from
surfaces of tunnels in localised areas (Figure 2-6) characterises rockbursts (Stacey and Ortlepp, 2001).

As the dynamic load imposed on shotcrete during seismic events is likely to be in excess of the strength of shotcrete liners, the key to successfully designing shotcrete liners to withstand dynamic load is to incorporate flexibility while still maintaining sufficient support pressure to stabilize the excavation after the event has passed. This approach may not satisfy serviceability requirements, but will minimise injuries.

Figure 2-6: Seismic loading on shotcrete (Stacey, 2001).

2.4 Shotcrete Properties and Standard Test Methods

This section defines the most important shotcrete properties and presents some methods of testing them. The basic concepts behind every testing method are discussed but the finer details of each testing method are not presented, as original codes are available for this purpose.

The following shotcrete properties are commonly for design purposes:

- Thickness of shotcrete;
• Adhesion between rock and shotcrete;
• Fibre density;
• Stiffness of the shotcrete;
• Uniaxial compressive strength of shotcrete;
• Shear strength of shotcrete
• Flexural strength of shotcrete; and
• Energy absorption and post yield behaviour.

The following standard testing specifications for shotcrete have been identified in literature:

• EFNARC;
• Swedish Standard SS 13 72 43, 1987;
• ASTM (American Society for Testing and Materials)

2.4.1 Thickness

Thickness of shotcrete can be tested by using depth pins or probing of the shotcrete while still wet. The shotcrete can also be applied in a grid pattern and the thickness gauged by the operator.

2.4.2 Adhesion

Adhesion is one of the controlling factors during failure of shotcrete linings (Baron, 1976 and Barrett and McCreath, 1995). Consequently, knowing the adhesive strength between rock and shotcrete is of great significance. Two adhesion test methods have been found in the literature, EFNARC and the Swedish Standard. Although many others are available only these two are presented as they represent the two basic methods of adhesion testing.

The Swedish Standard consists of drilling two circular holes surrounding an inner core through a layer of applied shotcrete as shown in Figure 2-7. The test apparatus is fixed to the inner core and the shotcrete-rock adhesion strength tested by pulling the shotcrete core from the rock.
The EFNARC test method consists of drilling a shotcrete/rock core through the interface and removing the core specimen (Figure 2-8). A direct tensile test is carried out on the specimen to determine the adhesive strength.
The factors influencing the adhesive strength of shotcrete are mostly related to rock type and cleaning of surfaces to be covered in shotcrete. For design purposes, dedicated test results may not always be available, but it is recommended that some testing is carried on all shotcrete surfaces and that the cleaning method used is recorded. This will allow the compilation of a database of realistic bond strengths that can be used for design.

The EFNARC specification recommends minimum shotcrete to rock bond strengths of 0.1 MPa and 0.5 MPa for non-structural and structural shotcrete respectively. The equivalent shotcrete to concrete bond strength specifications are 0.5 MPa and 1.0 MPa. Barrett and McCreath (1995) provide values of 0.5 MPa (poor adhesive strength) and 1.0 MPa (good adhesive strength).

### 2.4.3 Fibre density

The EFNARC standard recommends that fibre density should expressed in kg/m$^3$ (mass fibre / volume of shotcrete) after it has been sprayed. The required in-situ fibre density to meet the flexural strength, toughness and energy absorption requirements must be determined from testing. This is also used for quality control during routine spraying. Fibre density can vary between 1 kg/m$^3$ and 8 kg/m$^3$ for polypropylene fibre and between 30 kg/m$^3$ and 75 kg/m$^3$ for steel fibre. Fibre density is measured during preparation of shotcrete and by extracting and weighing fibres from crushed test samples. Methods for determining fibre density from fresh and hardened samples are specified in the EFNARC standard.

It should be noted that fibre losses of up to 50% could occur during spraying under difficult conditions.

### 2.4.4 Stiffness

The elastic properties of shotcrete are important when designing a shotcrete arch or beam support system. Since shotcrete may function as a beam when applied as a rock support system the flexural stiffness is required. The flexural stiffness may differ from the stiffness tested in uniaxial compression or tension as the shotcrete is subjected to both compression and tension at the same time. Hence the flexural stiffness tests are carried out on moulded shotcrete beams of standard dimensions depending on the code.

Malmgren (2004) presents the following equation to calculate the flexural Modulus of Elasticity for shotcrete beams:
\[ E_c = \frac{23}{1296} \times \frac{F_{cr} \times L^3}{\delta_0 \times I} \]

Where:

- \( F_{cr} = \text{the first crack stress} \)
- \( L = \text{the span length of the beam} \)
- \( \delta_0 = \text{mid span deflection of the beam} \)
- \( I = \text{the moment of inertia of the beam about the central axis or:} \)
  \[ I = \frac{b \times h^3}{12} \text{ for rectangular beams with } b \text{ and } h \text{ being the width and depth of the beam respectively.} \]

Flexural stiffness testing in the standard concrete testing codes such as EN 6784 and the ASTM codes can be used for testing the elastic modulus of shotcrete.

### 2.4.5 Uniaxial Compressive Strength

The uniaxial compressive strength (\( \sigma_{sc} \)) of shotcrete is one of the most widely used shotcrete properties and is valuable as an indicator of shotcrete quality as well as input for shotcrete design, in spite of its limitations.

The compressive strength of shotcrete can be tested using cube tests as for normal concrete or by coring samples of shotcrete. The major limitation of using cube tests is that dry mix shotcrete cannot be placed in concrete moulds and this method of testing is therefore unsuitable. The application of shotcrete also has a major influence and merely pouring the shotcrete mix into a mould would not give representative samples. Based on these limitations, the preferable method of testing shotcrete is by coring samples from representative panels of shotcrete.

Samples may be cored from specially prepared shotcrete panels or from shotcrete applied to a rock face. The following are some of the various specifications available for preparing shotcrete test panels.

- SANS Method 865
- European Specification for Sprayed Concrete (EFNARC);
• ASTM C31 Practice for Making and Curing Concrete Test Specimens in the field; and

The compressive strength of shotcrete typically ranges from 15 MPa (low strength, poor quality) to 60 MPa (high strength). Shotcrete mix designs are typically specified by compressive strength and the most commonly used mix strengths are 30 MPa and 40 MPa.

Testing of Production Shotcrete

The hardened shotcrete can be cored, trimmed and capped before being tested provided that the shotcrete layer is thick enough (approximately 75 mm). Care should also be taken to use a height to diameter ratio of between 1 and 2. Normally cores are removed and tested 28 days after spraying but different curing times may be specified subject to project requirements.

Testing of Prepared Samples

Shotcrete panels or boxes can be filled with shotcrete during the shotcrete process. These panels should have the same orientation as the rock being covered with shotcrete i.e. vertical or overhead. After hardening the shotcrete panels may be removed from their moulds and tested. The dimensions of these panels would depend on the test specification used. The following codes are available for coring from test panels:

• EN 7034
• SANS Method 865:1994

The following codes are available for compressive strength testing of cored samples:

• SANS Method 865:1994
• EN 4012

Table 2-1: Conversion Factors to Equivalent Cube and Cylinder Strengths (EFNARC, 1996)

<table>
<thead>
<tr>
<th>Height/Diameter ratio of core</th>
<th>Cube Factor</th>
<th>Cylinder Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>2,00</td>
<td>1,15</td>
<td>1,00</td>
</tr>
<tr>
<td>1,75</td>
<td>1,12</td>
<td>0,97</td>
</tr>
<tr>
<td>1,50</td>
<td>1,10</td>
<td>0,95</td>
</tr>
<tr>
<td>1,10</td>
<td>1,07</td>
<td>0,93</td>
</tr>
<tr>
<td>1,25</td>
<td>1,03</td>
<td>0,89</td>
</tr>
<tr>
<td>1,00</td>
<td>1,00</td>
<td>0,87</td>
</tr>
<tr>
<td>0,75</td>
<td>0,88</td>
<td>0,76</td>
</tr>
</tbody>
</table>


2.4.6 Shear strength

A standard test method for determining the direct shear strength could not be found in the literature. The SANS 0100-1:1992 (p26) species the minimum design shear strength ($\sigma_{sc}$) as a function of the uniaxial compressive strength ($\sigma_{sc}$).

$$\sigma_{ss} = 0.75\sqrt{\sigma_{sc}} \text{ or } 4.75 \text{ MPa (lesser of)}$$

Barrett and McCreath suggest shear strength values of 6 MPa and 8 MPa, for 30 MPa and 40 MPa shotcrete. These values are slightly higher than that calculated using the SABS 0100-1:1992 standard (4.1 MPa and 4.7 MPa).

2.4.7 Tensile strength

A standard test method for determining the tensile strength could not be found in the literature. The SANS 0100-1:1992 (p124) specifies the minimum design tensile strength ($\sigma_{st}$) as a function of the uniaxial compressive strength ($\sigma_{sc}$).

$$\sigma_{st} = 0.23\sqrt{\sigma_{sc}}$$

Barrett and McCreath suggest direct tensile strength values of 1.75 MPa and 2 MPa, for 30 MPa and 40 MPa shotcrete. These values are marginally higher than that calculated using the SABS 0100-1:1992 standard (1.25 MPa and 1.45 MPa).

2.4.8 Flexural Properties

The flexural properties of shotcrete are important since they influence the design of a shotcrete lining. The flexural strength of shotcrete is tested using a beam test (Figure 2-9). Various specifications have been published on the procedure to carry out a beam test. The most notable differences are that the size of the beam varies and that the method of calculating the flexural strength varies. The following codes are available for beam testing of shotcrete:

- ASTM C 1018 (1997)
- JSCE-SF4
- NCA No. 7 (1999)
- Betongrapport nr 4 (Stålfiberbetong,1995)
- EFNARC 1999
Two major properties are of concern, the first crack stress and the post yield behaviour. The first crack stress is the point where shotcrete reaches primary failure, and if loading continues, the shotcrete progresses into the zone of secondary failure.

Knowing first crack stress enables the designer to perform the correct design calculation for the expected shotcrete failure mechanism. The post yield properties are important since they determine the shotcrete behaviour when subjected to large deformations such as those caused by squeezing ground, seismic loading and high stress levels in soft ground.

*Figure 2-9: Example of a Beam test (ASTM C 1018)*

**ASTM C 1018**

The ASTM standard presents as output a first crack stress and toughness indices. The toughness indices are a measure of the post yield behaviour of the shotcrete and are calculated
by dividing the area under the load deflection curve to various predefined deflections with that of the area under the stress strain curve at the first crack stress. The following parameters are calculated using the ASTM code:

**First-crack stress:** determine by placing a straight edge along the linear part of the load-deflection curve (Figure 2-10). The first-crack stress is then given by the equation:

\[ \sigma_{ASTM} = \frac{P_{cr}l}{bh^2} \]

*where:*

- \( P_{cr} \) = first crack load
- \( b \) = width of beam
- \( h \) = depth of beam
- \( l \) = length of span

**I₅:** determined by dividing the area under the load-deflection curve up to point C by the area under the load-deflection curve up to point A.

**I₁₀:** determined by dividing the area under the load-deflection curve up to point E by the area under the load-deflection curve up to point A.

**I₂₀:** determined by dividing the area under the load-deflection curve up to point G by the area under the load-deflection curve up to point A.

*Figure 2-10: Flexural toughness (ASTM C1018)*
Determining $I_5$, $I_{10}$ and $I_{20}$ is a way of quantifying the post yield ductility of shotcrete, a property that has significance influence on the effectiveness of shotcrete during seismic loading.

Bernard 2002 determined the following range of ASTM toughness values from tests on 62 different shotcrete mixes: $I_{10} = 3$ to 7 and $I_{20} = 4$ to 14.

ASTM1018 also specifies the calculation of residual strength factors as follows:

$$R_{5,10} = 20(I_{10} - I_5)$$
$$R_{10,20} = 20(I_{20} - I_{10})$$
$$R_{30,10} = 5(I_{30} - I_{10})$$

**JSCE-SF4**

The JSCE-SF4 standard emanates from Japan and uses the same beam size as the ASTM standard. The JSCE-SF4 standard makes use of an absolute toughness value unlike the toughness relative to the first crack stress used in the ASTM standard. The toughness value is therefore sample size dependant as it is not normalized with respect to first crack deflection.

In order to calculate the toughness of shotcrete, the following steps should be taken:

i. Calculate the value of $T_{JSCE}$ which is equal to the area under the load-deflection curve up to a midspan deflection of $L/150$ where $L$ is the length of the beam.

ii. The toughness factor is defined as the following:

$$\sigma_{JSCE} = \frac{T_{JSCE} \times L}{b \times h^2 \times \delta}$$

where:

$L = \text{the beam length}$
$b = \text{the beam width}$
$h = \text{the beam height}$
$\delta = L/150$
The parameter $\sigma_{JSCE}$ is essentially the average flexural strength over the entire load deformation curve and is also referred to as the equivalent flexural strength (Vandewalle, 1997). Bernard (2002) determined the following range of JSCE toughness values from tests on 62 different shotcrete mixes: $T_{JSCE} = 4$ to $25$ and $\sigma_{JSCE} = 1.2$ to $5$ MPa.

**NCA No. 7 (1999)**

The NCA specification uses the residual flexural strength at various deflections as toughness values. Unlike the ASTM specification the NCA specification does not require the calculation of the “first crack stress” and the toughness is therefore an absolute toughness but as it is calculated in terms of stress, toughness is not dependant on the size of the beam.

The toughness is defined into three classes depending on residual flexural strength at specified deflections, these are presented in Table 2-2.

**Table 2-2: Toughness Classes According to NCA No. 7 (1999)**

<table>
<thead>
<tr>
<th>Toughness Class</th>
<th>Residual flexural strength at deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.5 mm</td>
</tr>
<tr>
<td>0</td>
<td>Shotcrete without reinforcement</td>
</tr>
<tr>
<td>1</td>
<td>2.5 MPa</td>
</tr>
<tr>
<td>2</td>
<td>4.5 MPa</td>
</tr>
</tbody>
</table>
The Betongrapport Nr. 4 (Stålfiberbetong, 1995) specification uses a similar beam size as the ASTM Standard. The definition of the relative toughness indices is also used as in the ASTM code, but the definition of the “first crack stress” is less subjective. The following procedure is used to determine the “first crack stress” as shown in Figure 2-12:

i. Define $F_{cr}^*$ as the point on the load-deflection curve where the curve starts bending.

ii. Calculate the slope of the load-deflection curve between loads corresponding to 0.25 and 0.75 times $F_{cr}^*$ and call this line L1.

iii. Draw a parallel line to L1, at a distance of h/2000 to the right, where h is the depth of the beam and call this line L2.

iv. Define the point P where the line L2 intersects the load-deflection curve.

v. Defined $F_{cr}$ as the maximum load between the points P and $F_{cr}^*$. If $F_{cr}$ differs from $F_{cr}^*$ by more than 10%, repeat steps i to v with $F_{cr}^*$ set equal to $F_{cr}$.

vi. Define the deflection at point $F_{cr}$ as $dQ$ and draw line L3 parallel to L2 through the point $F_{cr}$.

vii. Where L3 intersects the x-axis is defined as the point $dR$.

viii. The “first crack deflection” ($d_{fr}$) is defined as $dQ$ minus $dR$.

ix. For calculation purposes the intersection of L1 with the x-axis should be used as the origin.

Once the value of $F_{cr}$ is determined, the toughness indices are calculated as in the ASTM code.
EFNARC 1996

The EFNARC specification is used in Europe at the moment and is currently favoured in South Africa. Testing is carried out on beams, cut from shotcrete panels with a length of 600 mm, a width of 125 mm and a depth of 75 mm. The span length is 450 mm, and just as for the other methods, the beams are loaded at third points.

The flexural strength is determined by drawing the load-deflection curve of the beam test (Figure 2-13). The straight line portion of the elastic region below a point representing 50% of the peak load is selected. A line is drawn to the right with an offset of 0.1 mm. The highest load before the line is selected as the peak load ($P_{0.1}$) and the peak strength is calculated from the following equation:

$$\text{Flexural Strength } \sigma_{\text{EFNARC}} = P_{0.1} \times \frac{L}{b \times d^2}$$

where:

- $b$ = the width of the beam
- $d$ = the depth of the beam
- $L$ = is the span length

Figure 2-12: First Crack Load (Betongrapport Nr 4, Stålfiberbetong, 1995)
The EFNARC flexural strength classes are listed in Table 2-3

**Table 2-3: Flexural strength classes (EFNARC)**

<table>
<thead>
<tr>
<th>Minimum flexural strength (MPa)</th>
<th>STRENGTH CLASS</th>
<th>C24/30</th>
<th>C 36/45</th>
<th>C 44/55</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam flexural strength</td>
<td></td>
<td>3.4</td>
<td>4.2</td>
<td>4.6</td>
</tr>
</tbody>
</table>

*Figure 2-13: Flexural Toughness (EFNARC)*

The residual flexural behaviour is presented as the flexural stress at 0.5 mm, 1 mm, 2 mm and 4 mm. Based on each of these stresses the shotcrete is assigned into classes. The class divisions are presented in Figure 2-14 and Table 2-4.
Bernard (2002) determined the following range of EFNARC toughness values from tests on 62 different shotcrete mixes: Flexural strength = 4.2 MPa to 7.9 MPa, Residual strength$_{0.5\text{mm}}$ = 1.1 MPa to 5.0 MPa and Residual strength$_{3.0\text{mm}}$ = 0.35 MPa to 4.9 MPa.
2.4.9 Energy Absorption

Energy absorption is a property that determines the resistance of shotcrete against dynamic or large strain loading and can be tested using the methods specified by the EFNARC code and ASTM C1550.

EFNARC Square panel test

A test panel is prepared 600 mm long by 600 mm wide and 100 mm deep (Figure 2-15). During the test the panel is supported along its four edges while it is loaded in the centre by a 100 mm by 100 mm punch moving at a constant rate. The test is terminated when a deflection of 25 mm is reached and the load deflection curve drawn. In addition, the absorbed energy-deflection curve is also drawn, an example of which is presented in Figure 2-16. The toughness is classed according to the absorbed energy at 25 mm deflection and the classes are given in Table 2-5.

![Figure 2-15: Plate Test Set-Up](image-url)

**ASTM C1550 Round panels**

An alternative round panel has been introduced for more reliable testing of energy absorption. This is also known as the round determinate panel (RDP). The advantage of this test method is that a determinate crack pattern is always formed. Three cracks radiate from the centre of the panel to the boundary in between the supports (Figure 2-17). The test is invalid if the three typical cracks do not form.
Round panel tests typically produce lower energy absorption values than square (EFNARC) panels for the same mix. Bernard (2002) found that the Energy absorption for round ASTM panels ranged between 100 J and 1000 J for fibre reinforced shotcrete (22 J for unreinforced shotcrete).

2.4.10 Conclusions on shotcrete material properties

It is important specify the thickness of shotcrete for design purposes and measure the thickness of application for quality control. This is usually carried out in South African mines. Measurements of shotcrete thickness were taken during the underground shotcrete monitoring (section 4).
Adhesion tests are rarely carried out in South African Mines, since these are time consuming and difficult to carry out. During the underground shotcrete trials, some adhesion tests were carried out (section 4 and volume II, Appendix B2).

Fibre density is an extremely important shotcrete property, since it has a significant influence on the flexural strength. Fibre losses can be expected during underground spraying and this should be taken into account when specifying the mix design and it is important to measure fibre density in-situ for quality control purposes. Measurements of fibre density were carried out during the underground monitoring (section 4) and as part of the laboratory testing (section 6) programmes.

Shotcrete mix designs are invariably defined by the Uniaxial compressive strength. This parameter has very little influence on the flexural strength and energy absorption of shotcrete. However, it is a very simple test to execute and can be used reliably to measure the quality of application shotcrete. UCS tests were carried out during the underground monitoring (section 4) and laboratory testing (section 6) programmes.

Panel tests are more commonly used than beam tests in South African mines. This is largely due to the small deflections in standard beam tests which are considered unsuitable for application in South African Mines. The EFNARC test method is used more frequently than the ASTM method, mainly because an EFNARC testing facility has been available for longer and rock mechanics practitioners are therefore more familiar with the method.

Bernard (2002) carried out tests on 62 shotcrete mixes (more than 360 beams and 360 panels) and concluded that the most reliable method for determining post crack performance is the centrally loaded round panel (ASTM C1550). This test method provided the lowest variability in post crack performance results, which is largely due to the formation of a determinate crack pattern. Bernard also reported that the beam test methods display low reliability compared with panel test methods. He found beam tests to be most expensive to produce, since they require careful cutting prior to testing.

Panel and beam tests were conducted during both the underground monitoring and laboratory testing programmes (section 6). Further analyses, using yield line theory, of the EFNARC square and ASTM round panel tests results, are reported in section 7.5.

2.5 Design Philosophies

A design philosophy must be selected when carrying out design work.
This section presents the formal design philosophies found in literature and discusses their key attributes.

### 2.5.1 Observational Method

Peck (1969) presented the “observational approach” consisting of the following steps:

- Exploration sufficient to establish at least a general nature, pattern and properties of the deposits, but not necessarily in detail;
- Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions. In this assessment geology often plays a major role;
- Establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions;
- Selection of quantities to be observed as construction proceeds, and calculation of their anticipated values on the basis of the working hypothesis;
- Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions;
- Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis;
- Measurement of quantities to be observed and evaluation of actual conditions during construction; and
- Modification of the design to suit actual conditions.

Although this method is not often followed formally in shotcrete support design, it is presented here as a guide to approaching shotcrete support design.

### 2.5.2 New Austrian Tunnelling Method (NATM)

The New Austrian Tunnelling Method is based on the observational method and the ground response curve originally published by Fenner (1938) and Pacher (1964). The philosophy used when applying the NATM is to measure excavation closure until the optimum moment to apply support is reached (Figure 2-1). The level of support is then adjusted until closure stops. The timing of support is based on the ground response curve and the support response curve.
Since neither of these can be estimated accurately before construction, first estimates are made of the timing and level of support, but adjustments are made accordingly during construction.

The concept of allowing the ground to partially stabilise itself is not new. The originators of the NATM simply attempted to formalize the application thereof in tunnelling design.

### 2.5.3 Norwegian Tunnelling Method (NTM)

The Norwegian Tunnelling Method (Harold et al, 1983) was developed for hard rock tunnels in Norway and is roughly based on empirical principles. The underlying assumption of the NMT is that the excavation does not require support to maintain overall stability. The purpose of support is simply to stabilize any kinematic failure mechanisms that may occur.

Since the size of key-blocks depend on the rock mass joint such as joint spacings, lengths and orientations etc., the amount support is determined by the Q value of the rock mass. To aid the empirical design, a database of support applications was constructed and the Q-value was correlated with support levels that worked.

### 2.6 Shotcrete Support Design Methods

#### 2.6.1 Barton’s Empirical Chart

Barton et al (1976) first published their rock mass rating system or Q-value. This method was later upgraded and adjusted as the database expanded (Barton, 2002).

The basis for this empirical design method is the Q-value rock mass rating system and the Norwegian Tunnelling Method. A database was then developed listing the Q-values for excavations and the support installed. By correlating the level of support that was sufficient for stability for specific Q-values, a chart was produced where among other things the required shotcrete thickness could be read off (Figure 2-19).
This method is useful for providing a first estimate of the support requirements. It can be used to specify support requirements in routine mining applications, where the behaviour can be readily observed and monitored. Where the rock mass response can be expected to be more complex, it is necessary to carry out a more rigorous design (sections 2.6.2, 2.6.3, 2.6.4 and 2.7).

Stacey and Swart (2001) simplified Barton’s chart for mining applications (Figure 2-20). The excavation support ratio (ESR) defined for the Q-value was fixed for mining purposes.

Figure 2-19: Barton’s Shotcrete Design Chart (Barton, 2002)
2.6.2 Ground reaction curves and support interaction

The concept of ground reaction curves and support interaction has been discussed in section 2.1.2.

Hoek (1998) describes a method for determining the ground reaction curve in weak rock masses. The following equation was used to determine the ground reaction curve, which is appropriate for tunnels with a circular cross-section in a hydrostatic stress field and a uniformly applied support pressure:

$$ \frac{\delta_t}{d_0} = \left( 0.002 - 0.0025 \frac{P_i}{P_o} \right) \frac{\sigma_{cm}}{P_o} \left( 2 \frac{P_i}{P_o} - 2 \right) $$

where

- $\delta_t$ = Tunnel sidewall deformation
- $d_0$ = Original tunnel radius
- $P_i$ = Internal support pressure
- $\sigma_{cm}$ = Uniform applied support pressure

Figure 2-20: Shotcrete design chart (Stacey and Swart, 2001)
\[ \rho_o = \text{In situ stress} \]
\[ \sigma_{cm} = \text{Rock mass strength} \]

The rock mass strength should be determined from laboratory test results and the Geological Strength Index (GSI), which represents the rock mass characteristics. This equation takes into account the stress to strength ratio of the rock mass and incorporates failure of the rock surrounding the excavation. Hoek suggests that this method is suitable for obtaining a first estimate of the required support pressure to limit deformation, but recommends carrying out non-linear modelling to determine ground reaction curves for non-circular tunnel cross-sections in non-hydrostatic stress fields.

**Figure 2-21:** *Ground reaction curves derived using a closed formed solution and through finite element modelling (after Hoek, 1998)*

Hoek (1998) presents support pressure capacities for various shotcrete/concrete linings (Table 2-6) and determined the method which will be described in section 2.7.4. Hoek proposes using this method to provide a first estimate of the support requirements and then to carry out non-linear modelling to analyse the rock support interaction more rigorously.
Table 2-6: Maximum support pressure for shotcrete or concrete linings applied in tunnels with circular cross-section (after Hoek, 1998)

<table>
<thead>
<tr>
<th>Support type</th>
<th>Thickness - mm</th>
<th>Age - days</th>
<th>UCS - MPa</th>
<th>Curve number</th>
<th>Maximum support pressure $p_{l \text{max}}$ (MPa) for a tunnel of diameter $D$ (metres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete or shotcrete lining</td>
<td>100</td>
<td>28</td>
<td>35</td>
<td>20</td>
<td>$p_{l \text{max}} = 57.8D^{-0.92}$</td>
</tr>
<tr>
<td></td>
<td>150</td>
<td>28</td>
<td>35</td>
<td>21</td>
<td>$p_{l \text{max}} = 19.1D^{-0.92}$</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>28</td>
<td>35</td>
<td>22</td>
<td>$p_{l \text{max}} = 10.6D^{-0.97}$</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>28</td>
<td>35</td>
<td>23</td>
<td>$p_{l \text{max}} = 7.3D^{-0.98}$</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>3</td>
<td>11</td>
<td>24</td>
<td>$p_{l \text{max}} = 3.8D^{-0.99}$</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>0.5</td>
<td>6</td>
<td>25</td>
<td>$p_{l \text{max}} = 1.1D^{-0.97}$</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>0.5</td>
<td>6</td>
<td>26</td>
<td>$p_{l \text{max}} = 0.6D^{-1.0}$</td>
</tr>
</tbody>
</table>

Leach (1994, 1995 and 1998) and Speers and Spearing (1996) used finite difference modelling to derive ground reaction curves for overstressed tunnels in strong brittle rock (South African gold mines).

For a square excavation, Leach applied an internal support pressure, which is then gradually reduced and the tunnel closure is recorded as the support pressure is reduced. This approach has been extended using distinct element modelling of more realistic brittle rock masses in section 5. He then used the support pressure and flexural mode of failure analysis to derive closed form solutions for determining shotcrete capacity, which is described in section 2.7.5.

Speers and Spearing first allowed the modelled tunnels (square and elliptical) to deform with no support in order to identify the part of the excavation periphery that requires support pressure to prevent the model grid from collapsing. They then applied varying support pressures to those parts of the excavation to determine the support pressure versus deformation. The results show that significantly less support pressure is required to control deformation in a tunnel with elliptical cross-section. However, the support pressures required to limit deformation are extremely high and significant closure will occur without the use of extraordinary support.
Papworth (2002) briefly discusses the use of ground reaction curves and support interaction using support reaction curves derived from panel tests (section 2.4.9). This approach is illustrated in Figure 2-22. It is a very logical approach, but, unfortunately, methods for converting the panel test results to actual load deformation characteristics are not described or referenced. Methods of determining shotcrete capacity from panel tests will be discussed in section 2.7.7 and are explored further in section 7.4.

![Ground Reaction Curves]

**Figure 2-22: Schematic ground reaction curves and support interaction using shotcrete panel test results (Papworth 2002)**

### 2.6.3 Design for squeezing ground

The mechanisms of squeezing are described in section 2.3.2.

Hoek and Marinos (2000) proposed a method for predicting squeezing problems in heterogenous rock masses. They describe a rigorous method of estimating the strength of the rock mass and then suggest a method of analysis for providing a good first estimate of potential tunnelling problems due to squeezing conditions. A relationship between strain and the degree of difficulty in tunnelling through squeezing ground was proposed (Figure 2-23).

The method of analysis is based on the determination of the expected strain using closed form solutions (section 2.6.2) and rigorous non-linear modelling. The shotcrete capacity can be determined using further modelling or mode of failure analysis (section 2.7).
Figure 2-23: Approximate relationship between strain and the degree of difficulty associated with tunnelling through squeezing rock (after Hoek and Marinos 2000).

<table>
<thead>
<tr>
<th>Strain $\varepsilon$ %</th>
<th>Geotechnical issues</th>
<th>Support types</th>
</tr>
</thead>
<tbody>
<tr>
<td>A Less than 1</td>
<td>Few stability problems and very simple tunnel support design methods can be used. Tunnel support recommendations based upon rock mass classifications provide an adequate basis for design.</td>
<td>Very simple tunnelling conditions, with rockbolts and shotcrete typically used for support.</td>
</tr>
<tr>
<td>B 1 to 2.5</td>
<td>Convergence confinement methods are used to predict the formation of a 'plastic' zone in the rock mass surrounding a tunnel and of the interaction between the progressive development of this zone and different types of support.</td>
<td>Minor squeezing problems which are generally dealt with by rockbolts and shotcrete; sometimes with light steel sets or lattice girders are added for additional security.</td>
</tr>
<tr>
<td>C 2.5 to 5</td>
<td>Two-dimensional finite element analysis, incorporating support elements and excavation sequence, are normally used for this type of problem. Face stability is generally not a major problem.</td>
<td>Severe squeezing problems requiring rapid installation of support and careful control of construction quality. Heavy steel sets embedded in shotcrete are generally required.</td>
</tr>
<tr>
<td>D 5 to 10</td>
<td>The design of the tunnel is dominated by face stability issues and, while two-dimensional finite analyses are generally carried out some estimates of the effects of forepoling and face reinforcement are required.</td>
<td>Very severe squeezing and face stability problems. Forepoling and face reinforcement with steel sets embedded in shotcrete are usually necessary.</td>
</tr>
<tr>
<td>E More than 10</td>
<td>Severe face instability as well as squeezing of the tunnel make this an extremely difficult three-dimensional problem for which no effective design methods are currently available. Most solutions are based on experience.</td>
<td>Extreme squeezing problems. Forepoling and face reinforcement are usually applied and yielding support may be required in extreme cases.</td>
</tr>
</tbody>
</table>
Bosman et. al. (2000) analysed the squeezing behaviour at Hartebestfontein gold mine, where the rock is generally strong, but large deformations occur along bedding planes (Figure 2-24). (Refer to Figure 2-4a in section 2.3.2 for a description of this mechanism). A viscoplastic, displacement discontinuity model was used in which the material behaves elastically and all inelastic behaviour, including viscoplastic effects, is controlled by the presence of multiple interacting discontinuities. Bedding planes were represented as distinct persistent planes and potential rock failure surfaces are represented as multiple discontinuities forming triangular shapes. Shear slip on discontinuities is time-dependant, which allows for progressive redistribution of stresses near the openings. However, while the model results were encouraging and fairly realistic fracture behaviour was represented, the deformation predicted by the model was significantly less than that observed underground. Bosman et. al (2000) concluded that the model was not yet capable of reproducing this behaviour. This illustrates the difficulties in modelling this type of squeezing behaviour.
Figure 2-24: Modelling of squeezing behaviour in a gold mine tunnel (Bosman et. al. 2000)
2.6.4 Rock block analysis

Barrett and McCreath (1995) propose that the maximum size of a loose rock block, which a shotcrete lining should be designed to resist, can be estimated by a prism with side angles of 60° and a basal area defined by the spacing between tendons (Figure 2-25). Note that this is a worst case scenario, since no frictional or block interlock effects are taken into account and the analysis will therefore be conservative. Barrett and McCreath argue that frictional resistance can only stabilise a block if compressive tangential (clamping) stresses exist around the openings and because of the effects of blasting there could be stress relief in the skin of rock surrounding openings. It should be noted that in deep mines, clamping stresses are prevalent and provide a significant stabilising effect and this type of analysis will be extremely conservative. The deadweight load (ignoring frictional and block interlock effects) applied to the shotcrete for rectangular tendon spacing can be calculated as:

\[ W = \frac{\rho g ab^2 \tan \theta}{6} \]

where \( a, b \) are the larger and smaller tendon spacings,

\( g \) is the gravitational acceleration (9.8 m/s\(^2\)),

\( \rho \) is the density of the rock, and

\( \theta \) represents the side angles of a prism (60° for the maximum suggested by Barret and McCreath)

The support pressure is:

\[ w = \frac{\rho gb \tan \theta}{6} \]

Figure 2-25: Maximum size of loose rock block (after Barrett and McCreath, 1995)
Windsor (1998) suggests using tetrahedral blocks and shows the calculation of block volumes and weights. An alternative approach is to determine the maximum or upper 5 or 10 percentile block size that could fall out in between supports using a program such as JBlock (Esterhuizen, 2007), which uses statistical distributions of joint characteristics, joint networking, keyblock theory (Goodman and Shi, 1985) and limit equilibrium methods to form and analyse blocks (fall out between supports and support failure). The upper 5% of potential blocks can be analysed using the Rocscience software, Unwedge to determine the support pressure and load applied to the lining. Unwedge performs a full limit equilibrium analysis on rock blocks in the roof and walls of an excavation, which can have irregular geometries. This is particularly relevant when considering wall stability. Dunn (2007) used JBlock and Unwedge analyses for tendon design in tunnels. Shotcrete linings can be modelled in Unwedge, but the shotcrete model is limited and produces unconservative results. It is recommended that a more rigorous mode of failure analysis (section 2.7) should be used to determine capacity of shotcrete linings.

### 2.6.5 Numerical modelling of shotcrete linings

Several numerical modelling programs (eg FLAC, UDEC, Phase 2) allow the analysis of linings. The linings are modelled as beam elements, which are attached to the rock elements on the boundary of an excavation. Lining thickness and material properties are defined. The lining elements deform as the excavation deforms and the shotcrete will become loaded as a function of its stiffness. Irregular excavation geometries can be modelled, which will allow localised bending moments, compressive stresses and shear stresses to develop in the lining. The model resolution is of course limited by model run times.

Phase 2 version 7 and later versions also allow one to model the combined effect of shotcrete and different kinds of reinforcements. The Phase 2 analysis will give the distribution of the axial forces, which can be compressive or tensile depending on model properties, within each liner beam element as shown in Figure 2-26. It is also possible to plot strength envelopes for user-defined values of safety factor. These strength envelopes are based on the liner properties used in the analysis. The analysis results for each liner beam element are also displayed as individual data points on each plot, which allows the designer to easily visualise the stresses in the liner with respect to the strength envelope. Multiple safety factor envelopes can be plotted on one graph. Figure 2-27 shows the strength envelopes for safety factor values of 1, 1.2, and 1.4 for the cavern shown in Figure 2-26. Any data point that plots outside an envelope fails to meet the safety factor requirements of that particular envelope.
Figure 2-26  Phase 2 plot of the distribution of axial forces in the shotcrete beam elements used as support in a cavern.
Support capacity strength envelope for the shotcrete. Each data point represents a beam element in the shotcrete lining. Points outside an envelope have a FOS lower than the envelope FOS value.

In finite element, finite difference and discrete element codes, it is also possible to model the shotcrete as proper materials in the same way that the rock is modelled (eg Bernard and Pircher, 2001). However, the linings are very thin and very finely discretised elements are required to achieve meaningful results. The analyses are complex and detailed and are generally only carried out for research purposes.

2.7 Design of Shotcrete Support Using Mode of failure analysis

Several authors have derived equations for the various modes of failure listed in section 2.2 and these are described in the following sub-sections. Flexural failure appears to be the most commonly observed mode of failure and is covered more extensively in the literature. It is therefore covered in greater detail in this section than the other modes of failure.

It is useful to think of the demand imposed on the shotcrete and the capacity of shotcrete to resist the mode of failure. Failure of shotcrete is prevented by ensuring that the capacity is greater than the demand for all modes of failure. The demand imposed on the shotcrete is...
determined from the deadweight of a rock block, the required support pressure to resist deformation (section 2.6.2) or from numerical modelling (section 2.6.4). The capacity is determined using the material strength properties (section 2.4).

When comparing the demand and capacity it is necessary to apply a factor of safety to account for variability. For the design of reinforced concrete, the SABS 0100-1:1992 code makes use of partial safety factors for both loads and material properties that are probabilistically determined. This is more reliable and usually less conservative than the earlier approaches to reinforced concrete design. These partial safety factors effectively represent characteristic material strength values below which 5% of all possible test results would be expected to fail and characteristic load values above which not more than 5% of the spectrum of loading will lie (Allen, 1988). Note that Daehnke et. al. (2001) used the same probabilistic approach to account for the variability in the strength of timber elongates and block loads (fall out thickness) for stope support design in South African tabular hard rock mines (the statistical methods are the same, but partial safety factors were not determined). The percentile load and material values can be determined as follows:

\[ x = \mu \pm fs \]

where \( \mu \) and \( s \) are the mean and standard deviation of the parameter (normal distribution), and

\( f \) is the factor for the relevant percentile: \( f = 1.645 \) and 1.282 for 5% and 10% respectively.

The partial safety factor is simply \( x/\mu \). Higher partial safety factors are therefore specified for material strength properties and loads that have greater variability.

It is advantageous to use recognised partial material safety factors, because this saves having to carry out a large number of tests to determine the percentile values. The relevant material partial safety factors (SABS 010-1:1992) are listed in Table 2-7.

**Table 2-7: Material partial safety factors (SABS 0100-1:1992 p9)**

<table>
<thead>
<tr>
<th>Material property</th>
<th>Safety factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete flexural or compressive strength</td>
<td>1.5</td>
</tr>
<tr>
<td>Concrete shear strength</td>
<td>1.4</td>
</tr>
<tr>
<td>Concrete bond strength (note this is for steel bars)</td>
<td>1.4</td>
</tr>
<tr>
<td>Steel reinforcement</td>
<td>1.15</td>
</tr>
</tbody>
</table>
For all modes of the failure, the demand is determined from the weight \( W \) of a potential loose block or the support pressure \( w \) applied to the lining. The relationship between \( W \) and \( w \) is simply:

\[
W = w_{ab}
\]

\( W \) can be determined using the maximum size of a loose rock block (Barrett and McCreath, 1995) or using probabilistic and limit equilibrium methods (section 2.6.4). It is not considered necessary to apply a partial load factor when determining the maximum loose rock block, since the block size is likely to lie within the upper 5% of potential blocks that could form and frictional and block interlock effects are ignored. Nor is it necessary to apply a partial load factor if probabilistic methods have already been used to determine the loads.

When the support pressure \( w \) is determined using a ground reaction approach (section 2.6.2), it is possible to determine the upper 5% \( w \) using statistical distributions of rock properties and probabilistic methods. This is reasonably simple when using analytical methods, but significantly more complex and time consuming when using numerical methods. If no probabilistic analysis has been carried out, it is probably necessary to use a relatively high load safety factor when using this type of analysis, due to the substantial inherent variability in the rock mass characteristics and rock mass behaviour.

The following generic parameters are used for this section:

- \( W \) = load applied to the shotcrete (N),
- \( w \) = uniformly distributed load or support pressure applied by shotcrete (N/m\(^2\) or Pa)
- \( A \) = adhesive force (N) (subscripts c and d refer to capacity and demand respectively)
- \( T \) = direct shear force (N) (subscripts c and d refer to capacity and demand respectively)
- \( V \) = punching shear force (N) (subscripts c and d refer to capacity and demand respectively)
- \( M \) = bending moment per unit length of sections (Nm/m) (subscripts c and d refer to capacity and demand respectively)
- \( \rho \) = density (kg/m\(^3\)) (subscripts s, f and fm refer to shotcrete, fibre and fibre material (steel or plastic) respectively)
\[ \sigma = \text{strength (N/m}^2) \text{) (subscripts sa, ss, st, sf, sc, f, and m refer to shotcrete adhesive bond, shotcrete shear, shotcrete diagonal tensile, shotcrete flexural, compressive, fibre tensile and mesh tensile strengths respectively)} \]

\[ a, b = \text{larger and smaller tendon spacings (m) respectively,} \]
\[ x = \text{width of the tendon faceplate,} \]
\[ h = \text{thickness of shotcrete (m).} \]

**Figure 2-28: Generic parameters for mode of failure analysis**

### 2.7.1 Adhesion failure

Barrett and McCreath (1995) used the adhesion model proposed by Fernandez-Delgado et al (1981, 1976) to analyse adhesion failure. It must be noted that adhesion failure does not imply failure of the shotcrete, but simply makes the punching shear and flexural failure mechanisms kinematically possible (section 2.2.1).

**Demand**

The demand is simply the load imposed on the shotcrete lining:
Capacity

The capacity of a shotcrete lining to resist de-bonding \((A_c)\) for a rectangular pattern is (Figure 2-29):

\[
A_c = 2(a + b)\sigma_{sa} z_a
\]

where \(\sigma_{sa}\) is the adhesive strength of shotcrete (section 2.4.2), and

\(z_a\) is the adhesive bond length, defined as the distance from the perimeter of the panel (in the plane of the lining) over which the adhesive forces act (Figure 2-29). Adhesive bond lengths are 30 mm for relatively poor adhesive strengths of 0.5 MPa to 1.0 MPa (Hahn and Holmgren, 1979) and 50 mm for relatively good adhesive strengths of 1.0 MPa to 2.0 MPa (Fernandez-Delgado, 1981). SABS:0100-1:1992 specifies a partial safety factor of 1.40 for bond strength, although this is for steel bars in concrete. The shotcrete rock bond strength is likely to be more variable.

\[\begin{align*}
A_d &= W = \frac{\rho gb^2}{2\sqrt{3}} = wab \\

2.7.2 \text{ Direct shear failure} \\

Barrett and McCreaht (1995) propose that direct shear failure should be determined using the largest block that can be formed between tendons. The shear resistance is provided by the

Figure 2-29: Adhesion model modified from Barrett and McCreaht (1995)
shear failure surface formed within the shotcrete along the perimeter of the block (Figure 2-30). It is possible for smaller blocks to create shear loads on the lining, but these will always be less likely to fail, since the demand is proportional to the area or volume of the block, while the capacity is proportional to the perimeter of the block.

**Demand**

The demand ($T_d$) is simply the load imposed on the shotcrete lining (Figure 2-28):

$$T_d = W = \frac{\rho ab^2}{2\sqrt{3}} = wab$$

**Capacity**

The capacity ($T_c$) of a shotcrete lining to resist direct shear for a rectangular pattern is (Figure 2-30):

$$T_c = 2(a + b)h\sigma_{ss}$$

where $h$ is the thickness of the shotcrete, and $\sigma_{ss}$ is the shear strength of shotcrete in direct shear. Barrett and McCreath suggest values of between 1.0 MPa (8 hour strength) and 8.0 MPa (28 day strength). Direct shear strength tests can be conducted on shotcrete. SABS 0100-1:1992 specifies a minimum design strength of $\sigma_{ss} = 0.75\sqrt{\sigma_{sc}}$ or 4.75 MPa (lesser of). A partial safety factor of 1.40 is specified for shear strength.

![Diagram](image)

*Figure 2-30: Direct shear model modified from Barrett and McCreath (1995)*
2.7.3 Punching shear (diagonal tensile) failure

Barrett and McCreaith used the method defined empirically by the Canadian Reinforced Concrete Design Code (CSA Standard CAN3-A23.3-M84) to analyse punching shear failure. Adhesion failure (section 2.7.1) must take place first, before this mode of failure is kinematically possible.

Demand

The demand \( V_d \) is the load imposed on the shotcrete lining (Figure 2-28), however, in this case area of the face plate is removed, since the failure will occur beyond the face plate:

\[
V_d = w(ab - x^2)
\]

where \( x \) is the width of the faceplate.

Capacity

The capacity \( V_c \) of a shotcrete lining to resist punching shear for a rectangular pattern is (Figure 2-31):

\[
V_c = 4(x + z_{st})\sigma_{st}
\]

where \( \sigma_{st} \) is the diagonal tensile strength of shotcrete. Barrett and McCreaith suggest values of between 0.75 MPa (8 hour strength) and 2.0 MPa (28 day strength). SABS 0100-1:1992 specifies a minimum design strength of \( \sigma_{ss} = 0.23\sqrt{\sigma_{sc}} \). A partial safety factor of 1.50 is specified for tensile strength.

\( z_{st} \) is twice the distance from the edge of the faceplate to where the failure surface terminates. Failure is assumed to occur on an equivalent vertical plane defined by \( z_{st}/2 \), and \( z_{st} \) is determined as follows:

\[
z_{st} = \sqrt{\left(\frac{x}{4}\right)^2 + \left(\frac{V_d}{4\sigma_{st}}\right) - \left(\frac{x^2}{4}\right)}
\]

Note that if no face plates are used, the parameter \( z_{st} \) is non-zero.
2.7.4 Compressive failure

Brady and Brown (2006) and Hoek (1998) discuss the demand imposed on shotcrete under compression.

**Demand**

In section 2.6.2 the determination of a ground reaction curve in terms of support pressure versus the deformation of a tunnel is discussed. This can be derived using a closed form solution for a tunnel with a circular cross-section or through numerical modelling for more typical tunnel cross-sections.

Alternatively, the lining can be modelled using beam elements (section 2.6.5) attached to the inside of an excavation model, which enables the determination of compressive stress in each segment of the lining in response to tunnel deformation.

*Figure 2-31: Punching shear model modified from Barrett and McCreath (1995)*
Capacity

Hoek (1998) presents support pressure capacities for various shotcrete/concrete linings (Table 2-6) determined using the following equation from Brady and Brown (2006:p577):

\[ w = \frac{a_c}{2} \left[ 1 - \frac{(d_o - h)^2}{d_o^2} \right] \]

where \( h \) = thickness of the shotcrete lining.

\( d_o \) = internal tunnel radius

This equation is based on the assumption that the circular excavation is hydrostatically loaded, the internal support pressure is uniform, no bending moments or shear forces are developed in the lining and the mode of failure is compression. It should be noted that in non-circular tunnels in non-uniform stress fields, bending moments and shear will be developed in the lining. Hoek proposes using this method to provide a first estimate of the support requirements and then to carry out non-linear modelling to analyse the rock support interaction more rigorously.

If the compressive stress in the lining is determined using beam elements in numerical models, the capacity is simply the uniaxial compressive strength of the concrete.

2.7.5 Flexural Failure (Demand - Classical methods)

After adhesion failure (section 2.7.1) has taken place, it is possible for flexural failure to occur. Bending (flexure) occurs when the shotcrete panel (or beam) is fixed and/or supported at the boundaries and a load is applied some distance away from the boundary or uniformly over the entire panel. Bending moments are induced in the panel and these are influenced by the boundary conditions (continuous, fixed, supported or free), loading conditions [point loads (PL) and uniformly distributed loads (UDL)] and the length and width of the panel. The capacity of a shotcrete panel to resist bending moments is a function of the shape of the cross section of the slab and the properties of the shotcrete and its reinforcement.

When dealing with slabs the moment demand and capacity are conveniently expressed in terms of moment per unit length of cross-section with units Nm/m.

The moment demand \( (M_d) \) for beams/panels can be determined as follows:

\[ M_d = \frac{wb^2}{\alpha} \]
where $\alpha$ is determined using elastic theory for different loading conditions, boundary conditions and the ratio of $a/b$.

Panels generally have two way spanning moments (bending occurs along the length and width of the panel) unless $a >> b$. Timoshenko and Woinowsky-Krieger (1959) provide a number of elastic solutions for rectangular panels under different loading and boundary conditions. The simpler the loading and boundary conditions, the lower the values of $\alpha$ and the higher the moment demand.

Table 2-8 lists the values of parameter $\alpha$ used by various authors. This demonstrates that the assumptions made by the designer can significantly affect the required shotcrete thickness and reinforcing. Since the shotcrete and bolts effectively form panels supported by four bolts, it is suggested that $\alpha = 31.25$ for $a = b$ and $\alpha = 24$ for $a/b = 1.5$. 
Table 2-8:  Moment demand (values of parameter $\alpha$)

<table>
<thead>
<tr>
<th>$\alpha$</th>
<th>Reference</th>
<th>Loading and boundary conditions, ratio a/b, position and comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>Barrett and McCreath (1995)</td>
<td>UDL, simply supported at the two ends, two sides free, rectangular panel (a &gt;&gt; b), one way spanning. Maximum moment at centre of panel (simple span moment)</td>
</tr>
<tr>
<td>24</td>
<td>Leach and Naidoo (2001)</td>
<td>UDL, fixed at two ends, simple beam. Maximum moment at centre of panel. The length is taken as either the height of the tunnel wall or the tendon spacing</td>
</tr>
<tr>
<td>16</td>
<td>Kirsten (1992)</td>
<td>PL, continuous, square panel (a=b), supported on four sides, two way spanning. Maximum moment at centre of panel (Represents the large scale panel tests, section 2.8)</td>
</tr>
<tr>
<td>32</td>
<td>Kirsten (1992)</td>
<td>UDL, continuous, square panel (a=b), supported on four sides, two way spanning. Maximum moment at centre of panel (Represents the large scale panel tests, section 2.8)</td>
</tr>
<tr>
<td>31.25</td>
<td>SANS0100-1:1992 p 45</td>
<td>UDL, continuous, square panel (a=b), supported on four sides, two way spanning. Maximum moment at centre of panel.</td>
</tr>
<tr>
<td>24</td>
<td>SANS0100-1:1992 p 45</td>
<td>UDL, continuous, rectangular panel (a/b=1.5), supported on four sides, two way spanning. Maximum moment at centre of panel.</td>
</tr>
<tr>
<td>20</td>
<td>SANS0100-1:1992 p 45</td>
<td>UDL, continuous, rectangular panel (a/b=2.0), supported on four sides, one way spanning. Maximum moment at centre of panel.</td>
</tr>
<tr>
<td>29.4</td>
<td>SANS0100-1:1992 p 45</td>
<td>UDL, long edges free, short edges continuous, rectangular panel (a=b), supported on four sides, two way spanning. Maximum moment at centre of panel.</td>
</tr>
<tr>
<td>12.8</td>
<td>SANS0100-1:1992 p 45</td>
<td>UDL, long edges free, short edges continuous, rectangular panel (a/b=1.5), supported on four sides, two way spanning. Maximum moment at centre of panel.</td>
</tr>
</tbody>
</table>

The moment demand can also be determined through numerical modelling of elastic beam liner elements in numerical models (section 2.6.5).

2.7.6 Flexural Failure (Demand - Yield Line Theory)

Yield line theory was first published by K W Johansen (1962) in his Doctorate thesis and has subsequently become widely used as a design theory for reinforced concrete. Yield line theory may be applied to any material that exhibits elastic perfectly plastic behaviour and is therefore suited to reinforced shotcrete provided that adequate reinforcement is provided to ensure ductile behaviour. Yield line theory provides a simple method to calculate the ultimate load bearing capacity of shotcrete or alternatively the required plastic moment resistance for a given load on a shotcrete panel.
Yield line theory is based on the principle that internal work done by rotating yield lines must equal the external work done by the moving load. A yield line in this context is a line of cracked shotcrete serving as an axis of rotation for an un-cracked shotcrete panel. Only the energy absorbed by yield lines rotating is considered. The amount of energy absorbed by shear or elastic deformation is ignored for calculation purposes. In reinforced concrete design the error introduced by this assumption is very small but in shotcrete, the rock-shotcrete interface may absorb large amounts of energy in shear possibly leading to conservative results. Malmgren (2004) compared panel test results with predicted results and found that in general yield line theory provides conservative estimates of ultimate capacity in panel tests. The underestimation of ultimate load bearing capacity is speculated to be the result of arching in the shotcrete panel. In practice this arching effect may not always be relied upon and the results may be less conservative.

The derivation and detailed description of yield line theory is not discussed here but detailed information on the topic is available in Johansen (1962) and Kennedy and Goodchild (2004).

Thompson (2006) suggests that the response of shotcrete layers to impact may be simulated by yield line theory. This theory has been used to quantify the response of shotcrete layers restrained by different reinforcement systems. Thompson indicates the potential yield line pattern for a continuous shotcrete layer restrained by rockbolts (Figure 2-32. The results of Thompson’s preliminary simulations suggest that it is not possible for shotcrete to sustain the impact energies associated with the violent failure of brittle rock due to high stresses.

![Figure 2-32: Yield line pattern for areally loaded continuous shotcrete layer restrained by rock bolts (after Thompson, 2006)](image.png)
In the testing conducted by Morton et al.(2008) (also discussed in section 2.2.6), they concluded that the response of the composite sandstone shotcrete panel to punch loading, which induced flexural failure, was complex. They demonstrated that the crack patterns that develop are complex and representative of their underground observations and not the standard yield line patterns. This presumably means that the loading conditions, boundary conditions and the response of the shotcrete and rock are more complex than those represented by the more common yield line patterns observed in concrete beams.

The development of cracks and crack patterns were observed and analysed during the underground monitoring and are discussed in section 4.4. The crack patterns developed in EFNARC square and ASTM round panel tests are discussed in sections 7.4.1 and 7.5.1.

Yield line analysis is explored in more detail in Chapter 7 and a yield line solution for shotcrete and bolts is developed.

### 2.7.7 Flexural Failure (Capacity)

The moment capacity \( (M_c) \) or moment of resistance for beams/panels can be determined using elastic theory as follows:

\[
M_c = \frac{\sigma_{sf} I}{z_f} \quad \text{or} \quad M_c = \frac{\sigma_{sc} I}{z_f}
\]

where \( \sigma_{sf} \) is the flexural strength of shotcrete (section 2.4.8),

\( \sigma_{sc} \) is the flexural strength of shotcrete (section 2.4.5)

\( z_f \) is the depth of a point in the panel for which the flexural stress is relevant,

\( I \) is the moment of inertia or second moment of area, which represents the resistance provided by the shape of the cross-sectional area. For rectangular cross-sections:

\[
I = \frac{bh^3}{12} \quad \text{and for a unit length of cross-sectional area (b):} \quad I = \frac{h^3}{12}
\]

The maximum tensile and compressive stresses in an elastic panel occur at \( z_l = h/2 \) and \(-h/2\). Ignoring signs, \( M_c \) can be expressed as follows:

\[
M_c = \sigma_{sf} \frac{h^2}{6} = \sigma_{sc} \frac{h^2}{6}
\]

Since \( \sigma_{sf} \) is always much less than \( \sigma_{sc} \), the failure is invariably in tension. Vandewalle (1997), Barrett and McCreath (1995), Leach (1998) and Leach and Naidoo (2001) use elastic theory for their designs using beam and panel test results.
Plastic theory is used to design reinforced concrete panels. Once the shotcrete material capacity is exceeded, the reinforcement begins to carry the load. The stress no longer increases linearly from 0 to the maximum at \( z_f = h/2 \) and \(-h/2\) and becomes a function of the reinforcing material characteristics. Designs are carried out using plastic stress models (SABS 0100:1992 p 24 and Whitney, 1942). Kirsten (1992) uses plastic theory (Whitney 1942) to design fibre and mesh reinforced shotcrete.

**Moment Capacity from beam and panel test results**

Beam test results are more typically used for concrete and shotcrete design (Vandewalle, 1997, Barrett and McCreath, 1995 and Leach, 1998, Leach and Naidoo, 2001). Panel tests are not commonly used, but Leach suggests a method for determining moment capacity from beam and panel tests, which is discussed in section (2.7.8). The flexural material properties and test methods are described in section 2.4.8.

Vandewalle suggests that the first crack stress or modulus of rupture (\( \sigma_{ASTM} \)) in the ASTM 1018 test can be used to represent \( \sigma_{sf} \) for plain shotcrete, but uses a safety factor of 3 in his example, which is most likely due to the high variability in this parameter.

He recommends using the equivalent flexural strength, which is \( \sigma_{JSCE} \) in the Japanese standard (JSCE-SF4), for steel fibre reinforced shotcrete. This is effectively the average \( \sigma_{sf} \) across the load deflection curve and is therefore more representative of the contribution of fibres. Vandewalle does not use a factor of safety in this case, probably because the residual strength is more consistent. Tables are provided in Vandewalle (1997), which enable the determination of \( \sigma_{sf} \) for different types and densities of Dramix fibres.

Barrett and McCreath also do not recommend using \( \sigma_{ASTM} \) as \( \sigma_{sf} \) directly and propose the following for weld mesh or steel fibre reinforced shotcrete:

\[
\sigma_{sf} = 0.9 \frac{R_{10.5} + R_{30.10}}{200} \sigma_{ASTM}
\]

where \( R_{10.5} \) and \( R_{30.10} \) are the residual strength factors (ASTM 1018, section 2.4.8).

Leach determines \( \sigma_{sf} \) at a given deflection on the load deformation curve for beams and EFNARC panels, which represents the residual capacity at that point, which is discussed in more detail in section 2.7.8.
Moment capacity from fibre density and tensile strength

Kirsten (1992) proposed a method for determining moment capacity of shotcrete using the fibre tensile strength ($\sigma_f$) and fibre content ($c$) by volume. Since the fibres are not concentrated at the position of maximum tensile stress ($z_f = h/2$), a flexural crack will form and gradually all the fibres, in the outer half of the shotcrete panel, will deform and take load. These fibres provide the only residual tensile capacity. Kirsten assumes that fibres are randomly and homogeneously distributed throughout the cross-sectional area of the panel and uses 85% of the tensile strength of the fibres. An under reinforced condition is created, which means that the panel will tend to fail in tension rather compression. He proposed the following equation for determining $M_c$ using plastic theory (Whitney, 1973) as follows:

$$M_c = c\sigma_f \frac{h^2}{8.07}$$

Moment capacity for mesh reinforced shotcrete

Kirsten (1992) also provides a method for determining the flexural capacity of a mesh reinforced panel using plastic theory of reinforced concrete (Whitney, 1973). He assumes that a single layer of mesh is installed, which will lie roughly in the centre of the panel. In other words, the effective depth of the reinforcing, $d = h/2$. After the cracks begin to develop, the tensile strength is provided only by mesh reinforcement, which is effective at $d$. If the shotcrete is very thin and very strong heavy gauge mesh is used, it may be possible to create an over reinforced condition where the shotcrete panel can fail in compression, which is undesirable as this will result in a violent failure. Kirsten derived equations for both compressive and tensile stress.

For compressive stress, Kirsten uses $0.85\sigma_c$ to represent the compressive strength of shotcrete, (see section 2.4.5). The moment capacity in compression is determined as follows:

$$M_c = \sigma_c \beta (1 - 0.5\beta) \left(\frac{h}{2}\right)^2$$

where $\beta$ is the relative depth of the stress block and is dependent on the span to thickness ratio ($b/h$). $\beta$ is determined as follows:

$$\beta = \frac{257(b/h) + 230}{1000}$$

Kirsten determined the moment capacity in tension as follows:
\[ M_c = (1 - 0.5\beta)A_m\sigma_m \frac{h}{2} \]

In Kirsten’s test panels, the value of \( \beta \) ranges between 0.148 and 0.585, giving a range of 0.70 to 0.92 for the co-efficient \( 1-0.5\beta \).

The SABS 0100-1:1992 (pp24-25) equation for designing tension reinforcement, for \( d = h/2 \), reduces to:

\[ M_c = \frac{0.78}{1.15}A_m\sigma_m \frac{h}{2} \]

Note that this equation includes the material safety factor for steel reinforcing of 1.15. The coefficient 0.78 falls within the range used by Kirsten.

Vandewalle (1997) uses the following equation for designing mesh reinforcement \( (d = h/2) \):

\[ M_c = 0.9A_m\sigma_m \frac{h}{2} \]

The coefficient of 0.9 is within the range used by Kirsten.

### 2.7.8 Flexural Failure (Residual flexural capacity versus deflection - support reaction curve)

The load deflection curves for beams and panels could possibly be used to represent the support reaction (Figure 2-22, section 2.6.2). Since the length of beams and panels are not necessarily the same as the support spacing \((a,b)\), it is necessary to determine the equivalent in situ deflection \( \delta \) from the test panel deflection \( \delta_{pe} \) using the rotation \( \theta \), which is simply deflection divided by the length of the rotation arm:

\[ \theta = \frac{\delta}{\left( \frac{b}{2} \right)} = \frac{\delta_{pe}}{\left( \frac{b_{pe}}{2} \right)} = \frac{\delta_b}{\left( \frac{a_b}{2} \right)} \]

where \( \delta \) is the deflection,

\( b_{pe} \) and \( a_b \) are the EFNARC panel side length and beam length (distance between supports) respectively.

The \( \delta \) for an equivalent \( \theta \) is therefore:

\[ \delta = \delta_{pe} \frac{b}{b_{pe}} = \delta_b \frac{b}{a_b} \]
Leach (1998) and Leach and Naidoo (2001) used this concept to determine the required moment capacity from the results of a beam or an EFNARC panel test. They determined $w$ and $\delta$ from a ground reaction curve determined by non-linear modelling. In the absence of numerical modelling Leach suggests that the general design support pressure under deadweight loading ($w$) (South African industry accepted values eg. 50 kN/m$^2$) can be used for an anticipated or measured deflection. This alternative approach implies that the function of the shotcrete is not to resist support pressure, but must maintain sufficient support resistance to hold up rock blocks.

Leach determined $(\sigma_{sf})$ for beams as follows:

$$\sigma_{sf} = \frac{W_b a_b}{b_b h_b^2}$$

where $a_b$, $b_b$ and $h_b$ are the beam length, width and height.

Using the elastic solution provided by Wojtaszak (1936), Leach determines $(\sigma_{sf})$ for an EFNARC panel as follows:

$$\sigma_{sf} = \frac{1.28 W_{pe}}{h_{pe}^2}$$

where $h_{pe}$ is the thickness of the EFNARC panel (typically 75 mm)

Leach considered the shotcrete on a tunnel wall to act as a simple beam with fixed ends and determined the moment demand (section 2.7.5). He used elastic theory to determine the maximum flexural stress on the wall:

$$\sigma_{sf} = \frac{w b^2}{4 h^2}$$

He then found that the required $W_{pe}$ and $W_b$ could then be determined as follows:

$$W_{pe} = \frac{w b^2 h_{pe}^2}{5.12 h^2}$$

$$W_b = \frac{w b^2 b_b h_{pe}^2}{a_b 4 h^2}$$

Leach then uses the calculated $W_{pe}$ and $\delta_{pe}$ to check the material specification for different thicknesses as shown in Figure 2-33. The same principal is applied to beams.
Figure 2-33: Residual shotcrete capacity (after Leach and Naidoo, 2001)

Moment rotation curves have been used to characterize the residual response of beams and panels (Thompson, 2006, Bernard et al 2000, Armelin and Banthia, 1997) and are determined directly from the load-deflection curve for the particular test (Figure 2-34). $M_c$ is calculated from the loads at each point on the curve.

Figure 2-34: Moment rotation response for Plastic and Steel Fibre reinforced shotcrete (after Thompson 2006)

This concept is explored further in section 7.5.1.
2.7.9 Corrections for irregular excavation profiles and shotcrete cover techniques

Windsor proposes a method for determining an effective shotcrete lining thickness, which caters for irregular excavation profiles and shotcrete cover techniques (Windsor 1998 and Windsor and Thompson, 1999). The shotcrete cover techniques are described below and illustrated in Figure 2-35.

- **Type 1:** Coat the rough rock excavation surface with a minimum thickness to produce a rough coated surface;
- **Type 2:** Coat the rough rock surface with an even minimum thickness and partially fill the irregularities or “notches” to produce a rough coated and filled surface;
- **Type 3:** Fill the rough rock surface to a smooth surface defined by the “tips” of the rock projections to produce a relatively smooth filled surface;
- **Type 4:** Fill the irregularities to an F profile and then apply a minimum thickness cover over the tips to produce a smooth, filled and covered profile; and
- **Type 5:** Fill all overbreak and irregularities and continue with covering until a given excavation design surface is achieved to a specified excavation geometry.

\[
\begin{align*}
  h_1 &= h_c / \cos \alpha \\
  h_2 &= d_r \\
  h_3 &= d_a \\
  h_4 &= d_a + h_m \\
  h_5 &= d_a + h_{se}
\end{align*}
\]

where:

- \( h_c \) = average shotcrete coat thickness
- \( \alpha \) = average asperity angle
- \( d_r \) = average depth of shotcrete at asperity root
- \( d_a \) = average asperity depth
- \( h_m \) = average shotcrete cover above asperity tips
- \( h_{se} \) = average shotcrete cover above asperity tips
Referring to Figure 2-35, in South African mines, it is common to specify the cover technique 1, but in practice cover techniques 2 and 3 are often implemented. In some cases, where the application is very diligent, cover technique 4 is actually achieved. Cover technique 5 is rarely specified.

For cover technique 1, Windsor does not clearly define the average asperity angle, but since \( \cos \alpha \) has values between 0 and 1, \( h_1 \) must vary between \( h_c \) and infinity. It may be better to simply use \( h_c \).

Windsor uses the equivalent liner thickness to determine the capacity of the shotcrete under compressive stress (section 2.7.4). He derived equations for determining the equivalent elastic modulus, which considers the proportion of shotcrete and rock in the line of thrust for each cover technique (see Figure 2-36). This is then used to determine the equivalent stiffness of the shotcrete/rock liner. This is a fairly rigorous approach and less conservative than using the shotcrete thickness only. However, shear or tensile bond failures are not considered in the analysis.
Figure 2-35: Shotcrete cover techniques after Windsor and Thompson (1999)
Figure 2-36: A curved excavation surface showing the surface roughness and the equivalent liner thickness
Windsor (1998) also discusses failure mechanisms, in the context of structural design of shotcrete. He states that flexural failure cannot occur when cover techniques 1 to 3 are applied. This statement perhaps makes sense, if one considers the highly irregular sawtooth profiles illustrated in Figure 2-36, but flexural failure can certainly occur if cover technique 1 is used and the surface is not as rough.

Windsor (1998) also uses keyblock loading and analyses the direct shear mode of failure (section 2.7.2). He considers the shear resistance on a surface defined by the block perimeter multiplied by the equivalent lining thickness for each shotcrete cover technique.

2.8 Large shotcrete panel testing.

Three large scale panel test programmes were carried out in South Africa. Each of these programmes is briefly described and a summary of the findings is provided.

2.8.1 Tests carried out prior to 1992

Early investigations into the structural competence of mesh and fibre reinforced shotcrete were carried out on large shotcrete panels (Kirsten, 1992; Kirsten and Labrum, 1990).

These 1.6 m x 1.6 m shotcrete slabs were tested in the laboratory to investigate the comparative behaviour of differently reinforced shotcrete panels in terms of their bending characteristics under uniformly distributed or point loading. The setup is considered to be representative of the actual situation in which continuous shotcrete supports are secured by means of four regularly spaced bolts.

A steel frame was manufactured as shown in Figure 2-37 to Figure 2-39. The frame took a 1.6 m square panel and provided for bolt supports at the corners of the central 1.0 m square section. 100 mm square bearing plates attached to the bolt heads. This arrangement ensured continuity across the lines of support between the bolts and draws similarities to actual conditions.

Two loading cases were modelled in the tests: uniformly distributed loading (UDL) shown in Figure 2-38 and point loading in Figure 2-39. The design of the frame allows for the application of the uniformly distributed load by means of a hydraulically pressurised bag (Vetter bag) with a lifting height of 520 mm. Hydraulic pressurization of the bag limited the energy in the loading system and ensured that panel deflections could be tracked in a controlled manner beyond peak loading. A hand operated hydraulic jack with a stroke length of 150 mm could be used to apply appoint load at the centre of the panel. A bearing plate 100 mm square was placed between
the jack and the panel. The centre of the panel could be deflected to a maximum of 150mm for either of the loading cases.

**Figure 2-37:** Plan of panel testing frame (From Kirsten, 1992).

**Figure 2-38:** Section A-A through panel testing frame in Figure 2-37 illustrating UDL test (From Kirsten, 1998).
Pressure in the bag was measured by means of a transducer and the load in the jacks by means of a load cell. The deflections of the panel were recorded at the three positions: one at each of two diametrically opposite bolt positions and one at the centre of the panel as can be seen in Figure 2-40. The output signals from the various signals were used to produce continuous load-deflection/strain traces through the tests.

The design shotcrete mix comprised 37 per cent minus 6 mm river sand, 37.3% crusher sand, 15% ordinary Portland cement, 6% water, 1.7% silica fume, no accelerators. The shotcrete was either reinforced with fibres or mesh. Fibre reinforcement comprised 3% Dramix ZP steel fibre 30 mm in length by 0.50 mm in diameter. Mesh reinforcement, comprising diamond mesh with an aperture of 75 mm and a strand diameter of 3.1 mm, was placed in the middle of the panels.

Test panels were sprayed to thicknesses of 50 mm, 100 mm and 150 mm. Twelve test panels were sprayed to cater for the permutations of loading conditions, thickness and type of reinforcement. The panels were continuously spray-cured for three days after shooting, and subsequently moist-air cured until tested between 5 and 7 months later.

Under both loading cases and at small deflections the panels developed a tension crack in the top surface across the middle and parallel to the edges of the panel as shown in Figure 2-40. A second, similar tension crack develops shortly thereafter at right angles to the first crack as...
shown in Figure 2-41. As the test proceeded, more tension cracks developed in the bottom surface diagonally across the bolt holes (Kirsten, 1992, 1998; Kirsten and Labrum, 1990)

![Figure 2-40: The typical initial crack in the 100 mm thick fibre-reinforced panel subjected to point loading (From Kirsten and Labrum, 1990)](image)

At a further stage, compression cracks developed diagonally at the bolt heads. The mesh reinforcement failed across the cracks at large displacements, and was generally accompanied by crushing of the shotcrete on the compression side. The fibre reinforcement straddled the cracks and tended to pull out of the shotcrete, rather than snap.

![Figure 2-41: Typical ultimately cracked configuration of test panels (From Kirsten 1998)](image)
The main finding was that the mesh-reinforced shotcrete panels were superior to the fibre-reinforced shotcrete panels with regard to strength and ductility. The effective reinforcement in the fibre-reinforced panels was very much less than that in the mesh-reinforced panels. Fibre-reinforced shotcrete would be capable of sustaining load at prolonged deformation to a similar extent and would be equivalent in overall performance to mesh-reinforced shotcrete of similar thickness provided the fibre content is greater than 3 per cent by mass.

2.8.2 Testing carried out by the Shotcrete Working Group from 1994 to 1997

The Shotcrete Working Group was formed in 1994 to find the configuration of fibre reinforcement that would give shotcrete the required ductility. Ductility was defined as for this purpose to be the ability of a test panel to withstand not less than 50% of the peak load at a central deflection of 150 mm. Kirsten (1998) reported on the first series of tests conducted between 1994 and 1997.

The mix was designed by the Cement and Concrete Institute and comprised 78% river sand, 16.3% ordinary Portland cement, 4.3% unclassified fly ash (pozzfil), 1.3% condensed silica fume, CSF 90 and 0.5% dust suppressant and segregation preventive additive. A high strength shotcrete was obtained for some tests by using a very fine Andesitic lava aggregate instead of river sand. The mix was supplied in 30 kg bags at a moisture content of 5% that was increased during mixing to 10%.

Fibre reinforcement incorporated three different steel fibres (Dramix, Ferro Steel, Harex Steel), two different polypropylene fibres (Fibrillated PP and Monofilament PP). The fibre lengths ranged from 30 mm to 50 mm. Steel fibre content ranged from 1.4% to 4.3%. The polypropylene fibre was mixed at 0.4%, but in practice ranged from 0.2 to 0.9%. Two unreinforced and two diamond mesh reinforced panels were also tested for comparison.

Shotcrete was sprayed into 1.6 m square x 75 mm deep formers to create the shotcrete panels. More than 40 combinations were tested.

The same test arrangement was used as described in section 2.8.1. A vetter bag was used as before to apply a uniformly distributed load. The crack patterns developed during testing were similar to the previous testing (Figure 2-41).
For this test series only 40 mm and 50 mm long Dramix fibre and 30 mm, 40 mm and 50 mm long monofilament polypropylene fibre reinforcement are equivalent in ductility to mesh reinforcement and satisfy the ductility criterion that the ultimate load should be at least 50% of the peak load at 150 mm deflection. Unreinforced shotcrete possesses no ductility. Fibre lengths of 30 mm generally do not accord shotcrete the ductility that mesh does.

2.8.3 Testing carried out by the Shotcrete Working Group between 1999 and 2002

The Shotcrete Working Group started another test programme in 1999, which was unfortunately terminated early in 2001 due to withdrawal of funding. The test programme originally included 44 panel tests from 27 batches of shotcrete. In the end 27 panel tests from 7 batches of shotcrete were completed. The interim results of this test programme were documented in Keyter and Kirsten (2001).

Dry mix and wet mix batches were prepared. The dry mix comprised river sand (78%), ordinary portland cement (16%), pozzfill (4.5%) and silica fume (1.5%). A dust suppressant was added at the nozzle. The wet mix comprised sand (43%), stone (35%), ordinary portland cement (20%) and additives (2%).

The shotcrete was reinforced with Monofilament polypropylene fibre, Starprofiled polypropylene fibre, Dramix steel fibre and Harex steel fibre.

The same test apparatus as that described in section 2.8.1 was used. Four different loading and boundary conditions were applied:

- Simply supported with central point load
- Simply supported with distributed load
- Partially restrained along panel edge with distributed load
- Restrained along panel edges with distributed load

The following findings were made:

- The peak load carrying capacity and moment resistance of fibre is affected by the effective modulus of the composite material made up of sprayed concrete and fibre
• The testing arrangement with the edges of the panel restrained and with the panel loaded with a distributed load was found to be the most representative of what is generally observed in shotcrete tunnel linings which have undergone large deformation

• The starprofiled polypropylene performed better than monofilament polypropylene fibre

• The load at first crack as well as the overall load deformation behaviour of shotcrete panels reinforced with Dramix steel fibre outperformed the corresponding shotcrete panels reinforced with polypropylene fibre

• Some improvement in the overall load-deformation behaviour of fibre reinforced shotcrete was observed at higher fibre contents

• There was inherently no difference in the performance of wet and dry mix shotcrete

2.9 Dynamic testing of shotcrete

A large scale laboratory test facility was developed to simulate dynamic loading of shotcrete and mesh (Stacey and Ortlepp, 2001). This was done to ascertain the capacities of various types of wire mesh and shotcrete under dynamic loading. To approximate realistic field conditions as much as possible a drop weight was used. In a tunnel rockburst event, the dynamic loading imposed on surface support is in the form of a violent impact imposed by the rock mass and distributed across the surface support. The complete dynamic loading setup consisted of rockbolts and face plates, the surface support and the fractured rock mass surrounding the tunnel (provided the integrity of the fractured pieces is maintained) all of which contribute to the support resistance. These aspects are incorporated in attempting to simulate a rockburst event.

The constructed testing facility had to include:

• dynamic ‘impact’ loading

• shotcrete and mesh systems retained by rockbolts

• distribution of load onto the containment support through a ‘fractured rock mass’

• a ‘rock mass’ which would participate in the loading and deformation

• a large area of support, to take into account the areal continuity of containment support
Figure 2-42: Dynamic test loading facility (From Stacey and Ortlepp, 2001).

The dynamic testing loading facility is shown in Figure 2-43 above and has the following features:

- the shotcrete or mesh was hung from support beams by four rock bolts spaced 1m apart
- the size of the mesh or shotcrete panel was 1.6m × 1.6m, with the result that there is a 0.3m overlap beyond the rock bolt support on all sides of the panel
- anchors, grouted into the ground surface providing attachment points for the wires to account for the continuous nature of the shotcrete
- a simulated ‘rock mass’, consisting of three layers of 250mm × 250mm concrete blocks 100mm thick laid in direct contact with the shotcrete
- two drop weights with masses of 1050kg and 2700kg respectively were used to provide the dynamic loading
• the drop weight impacted onto a 40mm thick steel plate and the impact load was distributed onto the ‘rock mass’ by load distribution pyramid consisting of three layers of steel encased concrete blocks
• a drop height of 3.3m resulting in a maximum possible impact velocity of 8.1 m/s
• a maximum energy input of approximately 70kJ/m² was achievable

The materials subjected to the tests are as follows:
• welded wire mesh
• diamond (chain link) wire mesh
• special high yield meshes
• wire rope lacing with varying diameters
• shotcrete reinforced with welded wire mesh
• Dramix and monofilament polypropylene fibre reinforced shotcrete

The results of the test programme made it possible to have a comparative performance of surface support systems to be determined, i.e. the input energies and corresponding deformations were quantified. A summary of the results in terms of the centre deflection against total input energy is shown in Figure 2-43.
From the figure it is evident that unreinforced shotcrete has the poorest performance and weld mesh occupies the lower area of the plot. In almost all the tests, strands of the weld mesh broke and in some cases the welds broke as well, this being attributed to sharp edged steel face plates. The performance of diamond mesh was better than the weld mesh though the diamond mesh tended to unravel once a single wire strand had failed, spilling the ‘rock mass’ in the process. Fibre reinforced concrete was found to be approximately equivalent to diamond mesh in performance. Of the fibre reinforced shotcrete, Dramix steel fibres were slightly superior to the monofilament polypropylene fibres. Although the first drop of a weight on a fibre reinforced shotcrete panel is contained by the shotcrete, the second drop on the same panel destroys it. Fibre reinforced shotcrete on its own is therefore not an effective shotcrete component if subjected to repeated dynamic loading or dynamic loading after it has been cracked significantly by static deformation.

The dynamic loading tests further showed that the capability of the surface support system supports to absorb energy is enhanced considerably by the addition of wire rope lacing to the system. The performance of the weld mesh and diamond mesh were reversed by the addition of lacing. This is mainly because failure of the strand wires of the diamond mesh generally allowed
the ‘rock mass’ to spill through. Tests carried out on special yielding mesh and yielding mesh with wire rope lacing to evaluate yield capacity showed that the latter support withstood the maximum amount of energy without failing. This maximum amount of energy is considered to be representative of a severe rockburst. The results therefore represent the possibility of satisfactorily containing severe rockburst damage.

Quantification of the energy absorbing capacities of the various surface support systems, and their corresponding deformation limits are listed in Table 2-9.
Table 2-9: Energy absorbing capacities and deformation limits of surface support systems

<table>
<thead>
<tr>
<th>Shotcrete component/system</th>
<th>Input energy capacity (kJ)</th>
<th>Characteristic deformation limit (mm)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced shotcrete</td>
<td>6</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Weld mesh</td>
<td>10</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Diamond mesh</td>
<td>15</td>
<td>220</td>
<td></td>
</tr>
<tr>
<td>Special wire mesh</td>
<td>Varies, say 35</td>
<td>Varies, say 400</td>
<td>Mesh with commercially available or introduced yield capability</td>
</tr>
<tr>
<td>Weld mesh reinforced shotcrete</td>
<td>15</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>Dramix fibre-reinforced shotcrete</td>
<td>20</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Monofilament polypropylene fibre-reinforced shotcrete</td>
<td>15</td>
<td>120</td>
<td></td>
</tr>
<tr>
<td>Weld mesh and wire rope lacing</td>
<td>50</td>
<td>400</td>
<td>Wire rope lacing has some inherent yield capacity</td>
</tr>
<tr>
<td>Diamond mesh and wire rope lacing</td>
<td>35</td>
<td>300</td>
<td>Wire rope lacing has some inherent yield capacity</td>
</tr>
<tr>
<td>Special mesh and wire rope lacing</td>
<td>50</td>
<td>450</td>
<td>Wire rope lacing has some inherent yield capacity</td>
</tr>
<tr>
<td>Fibre reinforced shotcrete and wire rope lacing</td>
<td>35</td>
<td>170</td>
<td>Wire rope lacing has some inherent yield capacity</td>
</tr>
</tbody>
</table>

It can be concluded that a single surface support is unlikely to have all the desired characteristics to withstand dynamic loading, rather surface support systems consisting of combinations of various components will be likely to provide the desired characteristics.
3 Survey of shotcrete use in South African mines

As part of the initial data gathering phase of the project, a survey was conducted in early 2005 to determine the current shotcrete usage in the underground mining industry. A survey questionnaire was drawn up and distributed widely to rock mechanics practitioners working on mining operations in different commodities. The questionnaire records (a) general information about the operation and nature of the orebody, (b) specifics about how and where shotcrete is used in the operation, and (c) details about the person supplying the data (Appendix A).

Data was obtained from 22 operations and was captured in a database. Data from the above three categories (a-c) have been grouped separately as different sheets in Excel, and different tables in Access. The general operation information can be found in table/sheet “Operation”. The specific data about the application of shotcrete can be found in table/sheet “Application”. The details with regard to the person supplying the data is in table/sheet “Source”.

3.1 Data Integrity

An initial review of the data was done to determine its integrity and suitability for analysis. In general the questions were well understood and answered reasonably correctly. However, there are a few exceptions:

1. The main exception is with regard to the mining spans in the direction of the dip and strike of the orebody. Many data suppliers answered the question by incorrectly providing dip and strike of the orebody in degrees. The database was expanded to include fields for dip and strike in degrees, thereby accommodating the un-requested information. The missing data on the spans is not considered critical to the analysis.

2. One respondent (record ID 19 in the database) never tried to complete the questionnaire and gave only limited data in the “comment” field. Where possible, the comments were interpreted and some data fields completed.

3. Two respondents (ID 12 and 13) indicated that they did not currently use shotcrete. However, the second respondent also indicated that shotcrete was occasionally used as secondary support when a large excavation is developed. The recorded “<2%” application values were altered to 100% in order to fulfill the objective of the question, and the linear rate per month made 0 m in order to reflect the current status on the operation.
4. The difference in terminology from different mining methods (e.g. the massive mining block caving De Beers operations) may have created some confusion when answering the area of application questions. The data for one diamond operation (ID 2) was estimated on behalf of the respondent. Other potential misunderstandings are considered to have negligible impact on the data integrity.

5. There are slight differences in how respondents have interpreted the shotcrete type question (integration with other support components). Type options 3 and 4 are meant for conventional shotcrete with mesh and lace, or with tendons respectively. Some respondents may use fibre-reinforced shotcrete (“fibcrete) with the options 3 and 4. Two data source answers were changed to make the total percentage add up to 100%. The data was considered relatively robust as long as the potential ambiguity is understood.

6. The data cover four commodities. Table 3-1 shows the number of operations from each commodity that supplied data. Only one data source represents coal mining. The application of shotcrete on that one operation is unlikely to reflect the application across the coal industry. Three operations represent underground diamond mining, but all three are from large-scale De Beers operations. The data reflect the De Beers approach in South Africa, but are not representative of other small mining operations. The data for gold and platinum are slightly more comprehensive, but certainly not adequate for a detailed statistical analysis. The limitations and bias imposed by the small size of the dataset must be understood when considering the following analysis. The results are not considered statistically representative of the entire industry.

<table>
<thead>
<tr>
<th>Commodity</th>
<th>Number of data sets</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal</td>
<td>1</td>
</tr>
<tr>
<td>Diamond</td>
<td>3</td>
</tr>
<tr>
<td>Gold</td>
<td>11</td>
</tr>
<tr>
<td>Platinum</td>
<td>7</td>
</tr>
</tbody>
</table>

Table 3-1: Number of data sets used per commodity
3.2 Results

Different operations across the commodities and within commodities appear to have a different philosophy with regard to the systematic or adhoc application of shotcrete in rock support systems. Only slightly more than 50% of the operations prefer a more systematic approach (Figure 3-1), most commonly in the gold and diamond industry. Platinum mines tend to be shallow mining in relatively good rock and sometimes only require shotcrete sporadically as additional support in poor rock mass areas. Diamond mines tend to be much more systematic in the application of shotcrete because of the plastic behaviour of kimberlite and the need to keep kimberlite dry. Please note that in all the bar graph figures (Figure 3-1 to Figure 3-5), the actual % commodity contribution to each specific bar is biased by the number of questionnaires received and is not relevant. However, comparison of the relative size of a specific commodity contribution between category bars is useful. Also note that the data for the coal commodity are too limited for any coal-specific analysis, and are not considered further.

Shotcrete is used as a secondary support across all commodities (Figure 3-2). Such application is due to practical reasons. It cannot be easily combined with primary support because if it is applied too close to the development face it will be damaged by blasting. It is often used as tertiary support.
Figure 3-1: Bar graph showing the % application of shotcrete as an adhoc and systematic support. The relative contribution of each commodity to each bar is shown in colour.

Figure 3-2: Bar graph showing the % application of shotcrete as primary, secondary or tertiary type support. The relative contribution of each commodity to each bar is shown in colour.
Figure 3-3: Bar graph showing the % application of shotcrete to various mine areas. The relative contribution of each commodity to each bar is shown in colour.

The primary area of application of shotcrete support is in the haulages (Figure 3-3). It is also clear that relatively very little shotcrete is used on-reef near the production face in tabular mining, compared to more permanent off-reef infrastructure. Shotcrete tends to become a component of a rock support system that is meant to provide active or passive support over longer time periods than is required in the production stope.

The wet or dry application of shotcrete is split nearly evenly between operations (Figure 3-4). However, platinum mines appear to have a real bias towards the dry application of shotcrete.
Figure 3-4: Bar graph showing the % application of wet and dry shotcrete. The relative contribution of each commodity to each bar is shown in colour.

Figure 3-5: Bar graph showing the % application of different types of shotcrete support systems. The relative contribution of each commodity to each bar is shown in colour.

The data indicate that there is a preference towards steel fibre-reinforced shotcrete ("fibrecrete"), although the marginal majority of shotcrete support systems are used in
conjunction with tendon reinforcement, particularly on the platinum mines (Figure 3-5) with higher key-block loads. The data does suggest a relatively variable approach to the application of shotcrete within support systems across and within commodities, perhaps indicating the need for a clearer understanding of the benefits of shotcrete in different situations. The four main support systems on the questionnaire each contribute between 14% and 36% to total 95% of the types of applications. Less common support variations make up the remaining 5%.

The quantities of shotcrete used on the underground mining operations is estimated in the data by the linear metres of shotcrete applied per month (m/mth.; Figure 3-6). The majority of operations appear to operate at less than 300 m/mth, and most commonly around 100 m/mth. However, three operations (14% of the data records) use between 1000 and 1550 m/mth. Table 3-2 shows how the rate of application varies between commodities. The higher rates on gold and platinum operations probably reflects the larger number of development ends on these operations.

![Figure 3-6: Frequency histogram rate of application of shotcrete in underground mining operations.](image)
Table 3-2: Average shotcrete application rate per commodity

<table>
<thead>
<tr>
<th>Commodity</th>
<th>Linear metre per month</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal (1 mine)</td>
<td>100</td>
</tr>
<tr>
<td>Diamond</td>
<td>180</td>
</tr>
<tr>
<td>Gold</td>
<td>346</td>
</tr>
<tr>
<td>Platinum</td>
<td>330</td>
</tr>
</tbody>
</table>

3.3 Recommendations

A more detailed statistical analysis could be done if more data are obtained. In particular, more data from the coal industry are warranted.

Although the questionnaire is technically correct, some very minor alterations to the questionnaire would remove some misunderstandings by the data suppliers. The alterations could clarify questions regarding the mining spans and the type of support used in combination with shotcrete.
4 Underground monitoring of the interaction of shotcrete and rock

The interaction of shotcrete and the rock to which it is applied has been documented to some degree in shallow, statically loaded mining environments. One good example of such a study is by Barrett and McCreath (1995), where the authors suggest an analytical approach to the design of shotcrete under these conditions based on deterministic models. Under these environments loading of the shotcrete is driven by the movement of structurally defined blocks and wedges under gravity.

For the deeper stress-driven failure and dynamic environments very little has been documented on the interaction of shotcrete and rock. To understand the shotcrete-rock interface the mechanisms behind the failure of shotcrete under these conditions need to be closely investigated. This chapter is dedicated to the instrumentation and analysis of a series of underground test sites established with the goal of achieving exactly that.

4.1 Objectives of the monitoring program

The main objectives of the underground instrumentation monitoring programme were to

- Identify, establish and instrument suitable underground test sites that collectively cover high and low deformation quasi-static as well as dynamic loading conditions.
- Continuously monitor deformation, strain, dynamic ground motion, shotcrete crack propagation and shotcrete failure modes until site closure when no further stress changes are expected.
- Determine the driving stresses behind the measured deformations through numerical modelling
- Combine results to conduct thorough assessments.
- Based on the assessments describe the shotcrete-rock mass interface and the mechanisms of shotcrete failure.
4.2 Description of instrumentation used

The underground monitoring exercise required the use of various monitoring instruments at each of the chosen sites. The primary purpose behind the instrumentation installed was to measure sidewall deformations and to better understand the loading conditions present. The various instruments that were used are briefly described below.

4.2.1 Measurement of displacement

Single point and Multi point Borehole Extensometers (SPBX and MPBX)

These are rod type borehole extensometers. A typical rod extensometer consists of a reference head, usually installed at the collar of a drill hole, and one or more in-hole anchors, each of which is fixed in place at a known depth in the borehole. Aluminium rods extend from the in-hole anchor points to the reference head at the hole collar where rod displacements can be measured. As the rock deforms, the distances between the individual in-hole anchors and the reference head change. This allows for determination of distribution and rate of deformations in the rock mass intersected by the drill holes.

SPBX and MPBX measurements were taken manually with each site visit, once a week or once every two weeks depending on the site. These readings were generally adequate at sites where high deformation rates were recorded and their analysis has contributed greatly to this study. At sites with lower deformation rates (pillars D and E of South Deep site 2) the borehole extensometer readings were prone to some inaccuracy given the limitations of the manual measurements and the smaller deformations that were occurring.

Ground movement monitor (GMM) with continuous data capture

YieldPoint's DETECT GMM (Ground Movement Monitor), shown in Figure 4-1, is a high precision digital instrument comprising a long-range eddy current sensor and a digital temperature sensor. The GMM is in essence a digital version of the SPBX described above. The GMM sensor is usually attached to a 5/8” rock bolt which is anchored at a known distance into a 30 mm diameter drill hole. The rock bolt then replaces the function of the aluminium rod and in-hole anchor of a typical SPBX installation.

An on-board microcontroller provides automatic temperature compensation and readout can be made manually using a manual interrogation unit (MUI). The true advantage of the GMM
installation lies in its high accuracy and resolution and the fact that it can accommodate continuous data logging. The 125 mm travel GMM version used in this project provides an accuracy of 0.625 mm and a resolution of 0.0125 mm. Automated continuous data retrieval is achieved using YieldPoint’s “Sensor Logger for Underground” SLUG technology.

GMMs were purchased and installed at a later stage in the project (when they became available) and have undoubtedly contributed greatly to the analysis conducted. Continuous, high resolution readings made it possible to identify and analyse small and instantaneous changes in deformation.

Figure 4-1  YieldPoint’s GMM and manual readout unit

Laser target systems
A Hilti laser range finder was used to take manual readings of sidewall convergence during site visits. These measurements were carried out with the objective of obtaining crude displacement measurements at multiple locations not covered by other instruments. The device was secured to an instrument similar in design to a surveyor’s theodolite allowing both horizontal and vertical rotation. Changes in the distance measured between the instrument mounting point and fixed target points on a sidewall of interest were then measured. The setup was mounted either to the locomotive rails on the footwall of a tunnel or to a wall opposite the sidewall being monitored.

An initial prototype (Figure 4-2) was found to be unreliable and was later replaced by the more rigid design shown in Figure 4-3. This setup, although less accurate and reliable than the other instrumentation types used, provides a simple and cheap solution for monitoring deformation changes at many different points. One important consideration when using this type of setup is
that the mount point is usually not fixed under higher stress conditions. Movement of both the rail and the sidewall mount points was troublesome and often resulted in misleading results.

Figure 4-2  Initial prototype of the laser range finder

Figure 4-3  Example of the operation of the improved laser range finder

4.2.2 Measurement of strain

At some of the sites strain gauges were installed into the fractured rock mass behind the shotcrete to provide an indication of stress changes within this fractured ground and to correlate this with numerical modeling results. Strain gauges were installed at both the South Deep sites and at the Impala platinum site.

Geokon 4202 vibrating wire concrete embedment strain gages (VWSG) were selected because they could be easily installed in an intensely fractured borehole. SRK, in discussions with instrumentation experts including Ewan Sellers (CSIR Miningtek) and Peter de Haan (Terra monitoring), was advised against the use of CSIRO hollow inclusion (HI) cells or hard inclusion
vibrating wire stress meters since it is difficult to ensure proper seating of these instruments in an intensely fractured borehole.

A typical installation consisted of three separate gauges orientated differently or at different depths within the borehole. An example of such an installation is shown in Figure 4-4.

![Typical vibrating wire strain gauge installation](image)

**Figure 4-4**  *Typical vibrating wire strain gauge installation*

The VWSG installations produced unexpected results, which after much investigation were not extensively used in the analysis. Difficulties linked to strain measurement in a highly fractured rock mass were expected and the results provide a typical example of the problems that can and do occur.

- In many cases tensile strains were measured where compression would have been expected.
- Some sites measured sudden large jumps in tensile strain within a month or two after installation.
- At the South Deep installations, measured strains exceeded the operational limits of the Geokon 4202 strain gauges.
The exact mechanisms behind the results are complex, but are believed to be rooted in a difference in the material properties between the host rock and the grout into which the gauges are set.

The choice of instrument for this project was based on a detailed assessment of the pros and cons of each option. However, in light of the results obtained, it is advised that future attempts at strain measurement under similar conditions, hard inclusion type strain cells be considered. The elastic modulus of such an instrument is greater than that of the rock mass into which it is installed and stress can therefore be determined from the strain measurements and the known elastic modulus of the steel used. Some modifications may be required to overcome the problem of proper seating within a fractured rockmass.

4.2.3 Measurement of strong ground motion (Seismicity)

Peak velocity detectors (PVD)

The PVD, developed by CSIR Miningtek, is a portable instrument specially designed for recording strong ground motions. An example is shown in Figure 4-5.

![Installed peak velocity detector (PVD)](image)
The PVD is a battery powered stand-alone device with backed-up memory capable of storing up to 512 peak particle velocities. When the PVD has reached its capacity of 512 peak particle velocities, it continues to record ground motions, but stores only the highest velocities.

PVDs installed at the South Deep sites successfully recorded strong ground motions from nearby blasting activities. Seismic events resulted in smaller peak particle velocities (PPVs) and only a few were picked up by the PVD instruments. At the Mponeng 116 site a PVD was installed to record ground motions from seismic events, but the results were not satisfactory.

Mine seismic networks

The seismic system databases at the South Deep and Mponeng mines were used to identify seismic events, which caused ground motions recorded by the PVDs. The databases include date, time, location, local magnitude, seismic energy, seismic moment and several other seismological parameters for each seismic event. Dates and times of recorded seismic events could be matched with those of PVD recordings. PPVs were also estimated from the magnitude/seismic moment and distance, using an empirical relation.

4.2.4 Crack and fracture mapping

Borehole video camera

Borehole video camera footage was used at each of the sites to investigate the depth of, and changes to, sidewall fracturing over time. High humidity often resulted in poor video quality which limited the detail that could be investigated. Even though detailed fracture mapping was not possible (given the quality of the video) the obtained video footage was useful in identifying zones of changing fracture intensity.

Comprehensive photographic records

Photographs have been a dominant tool in the investigational process of this project. If done consistently and meticulously, the taking of photographs can provide the most insightful clues during the analysis stage.

It took some learning before a good database, capable of providing consistent results, was created. Much of the time spent at South Deep site 1 served as a learning process from which
improvements were identified and were then transferred to site 2. Based on the experience, the
creation of a good photographic database relies on the following ground rules:

- Use a high resolution camera with flash capability. When possible it is always better to
  use additional lighting.
- Allow enough time for the camera to acclimatise (defog) before taking photographs.
- The same person should ideally take the photographs for a given site. Consistency in
  photographic perception or the position from which photographs are taken makes later
  analysis significantly easier and more productive.
- Ensure that photographs are taken as perpendicular to the wall as possible. This
  accommodates better merging of photographs.
- Follow a set sequence of photographs which collectively cover the site in full before
  taking time to capture any specific changes noted on that day. This is crucial in
  maintaining a consistent overview of changes occurring at the site. Here it is more
  important to encompass the site as a whole – beware of taking close-up pictures
  focussing on specific aspects. This often leads to an incomplete understanding during
  subsequent analyses.
- Do not rely on the photographs alone. The importance of this cannot be stressed
  enough. Photographs provide a rather limited two dimensional view of the site being
  monitored. Three dimensional stereo photographs are always advisable, but even these
  are prey to deception by shadows and other effects. The best results were obtained
  when hand drawn sketches and notes were made at the time of the visits (of crack
  locations and progression etc.) and these were then used to guide the focus of analysis
  of the photographs.

4.3 Monitoring sites

Over the 3 year monitoring period 5 separate sites were identified and instrumented. Each of
the sites and their establishment is discussed in the sub sections that follow and a summary is
provided in Table 4-1. Detailed discussions of each of the sites can be viewed in Appendices
A1 through A5.
4.3.1 Site description

South Deep, 87 2W trackless Site 1

This site, established in July 2005, was situated in a trackless mechanised drift and fill operation that was in the final stages of extraction at the time of monitoring. The site lies on 87 Level at the 2-West Trackless section at South Deep Mine at a depth of about 2550 mbs. The location of this site, as well as the location of South Deep site 2 (which was an extension of site 1) is shown in Figure 4-6. The mining method incorporates drifting, benching and filling in a predetermined sequence.

Site 1 comprised a pillar (secondary drift) situated in brittle, massive conglomerate at 2550m below surface, which was to be extracted at the end of the mining sequence. The pillar measured 4m in height, 8m in width and 60m in length and a portion of it was sprayed with steel fibre reinforced shotcrete. High pillar stresses were evident by the presence of tensile fractures, which were dilating. Stresses in the adjacent pillars (secondary drifts) were high and were noted to increase significantly as the final drifts and benches were mined. This made the site ideal for the monitoring of shotcrete under quasi-static loading conditions where high deformations and extensive shotcrete damage is expected.

Instrumentation included two extensometers (2 point) to measure dilation and outward deformation, a vibrating wire strain gauge (VWSG) rosette to measure strain changes within the pillar and two peak velocity detectors (PVD’s) to measure horizontal ground motion. In addition, closure was monitored using a customised laser range finder from fixed points on an opposite wall. Results obtained from this device proved to be very inaccurate leading to the design and manufacture of a new device for use at site 2.

In November 2007, the site was rendered inaccessible when it was backfilled closed. Experience and data from this site provided opportunity to identify the pitfalls of such an instrumentation and monitoring program. The experience gained from site 1 was applied in the establishment of site 2 in January 2007.

South Deep, 87 2W trackless Site 2

This site, identified in the second half of 2006 and established in January 2007, is an extension of site 1. Situated close to site 1 (Figure 4-6), site 2 comprises several final safety pillars which are located around a tipping area (Figure 4-7). Steel fibre reinforced shotcrete was applied to
the sidewalls of the pillars using a wet shotcrete process. Four pillars (A, B, D and E) were successfully monitored between February 2007 and May 2008.

Electronic instrumentation included a combination of SPBX and MPBX extensometers, VWSG’s, PVD’s and laser targets using the laser range finding device. Ground motion monitors (GMM’s), not previously used at site 1, were incorporated at site 2 providing greatly improved resolution and sensitivity whilst also allowing for continuous monitoring through SLUG digital interface technology. Monitoring of outward deformation of pillar sidewalls was greatly improved using the GMM and SLUG technologies. Continuous VWSG measurement was also made possible through Yielpoint’s “Digiplucker” digital interface technology.

Guided by the lessons learnt at site 1, results from site 2 pillars have yielded results that have contributed greatly to the findings of this study.

**Mponeng Mine, 109 level**

Regional and local mine plans show the location of the 109 site (Figure 4-8 and Figure 4-9). Established at the end of 2005, the site comprises a 5m length of 4.5m x 4.5m crosscut at 3037m below surface on 109 level at the reef intersection. The site is 15 m above reef within the hangingwall Alberton lavas. Both sidewalls of the crosscut (east and west) were used to give two sites, each measuring 5m long by 3m high. The area was originally supported with wire mesh and lacing. The mesh and lacing was stripped over a selected 5 m length and was sprayed with steel fibre-reinforced dry shotcrete.

The planned mining layout was expected to cause a substantial stress increase in the tunnel sidewalls and large seismic events during the final stages of mining. Changes to the mining layout after initial site establishment resulted in the expected stress levels and seismicity not being experienced. The site was instead exposed to a reducing stress field as the crosscut was overstopped. Instrumentation focus at this site was reduced with more emphasis being placed on the monitoring of South Deep site 2. Final site instrumentation included laser target monitoring as well as bore-hole video scoping.

This site was closed in September 2007 and although yielding limited monitoring data, observations made suggest that the applied shotcrete performed well in the decreasing stress field.
Mponeng Mine, 116 level

In October 2006 this site was added to the monitoring program. The site is 3271 m below surface on 116 level in a highly seismically active area (Figure 4-8 and Figure 4-10). The site is located beneath a dyke dip pillar 90m in the pillar footwall. Mining has taken place close to the dyke and has resulted in associated seismicity. This location was chosen specifically to measure shotcrete performance under dynamic loading conditions. The area is heavily supported with 50mm steel fibre reinforced shotcrete covered with wire mesh and lacing as well as 38 tonne long anchors. As this site was chosen after it had been supported, the specifics of the applied fibre-reinforced shotcrete are not known.

Impala Platinum 14 shaft, 24 level

Established in March 2006, the site is a 3m x 3m footwall drive at 1200m below surface on 24 level (Figure 4-11). It is separated from the stoping above by a 7m middling. The drive is situated in a brittle 70 MPa strength spotted anorthosite. Wet shotcrete was sprayed on both sidewalls at an average thickness of 50mm. The shotcrete is not fibre reinforced.

It was originally expected that the ground surrounding this area would be extensively mined leaving a safety pillar just above the site. It was thus anticipated that extensive sidewall deformation would occur resulting in failure of the shotcrete.

In reality, due to mining and geological difficulties, much less of the ground has been mined and relatively low deformations of about 0.4 mm per year have been recorded by installed GMM’s. Installed VWSG rosettes have also delivered data but the interpretation these results has proven to be complex and misleading.

Due to the low deformations measured, this site has offered useful data in terms of the pre-failure performance of shotcrete (without fibre reinforcement) under quasi-static loading conditions where low deformation and limited shotcrete damage occurs.
Figure 4-6  Regional mine plan and enlarged view of both South Deep test sites
Figure 4-7  Test pillar detail at South Deep Site 2

Figure 4-8  Regional mine plan of the Mponeng 109 and 116 level test sites
Figure 4-9: Enlarged view of Figure 4-8 showing the Mponeng 109 level test site

Figure 4-10: Enlarged view of Figure 4-8 showing the Mponeng 116 level test site
Figure 4-11  Regional mine plan and enlarged view of the Impala test site
4.3.2 Site preparation

Rock mass condition

Before the application of shotcrete the condition of the rock mass at each of the sites was assessed using the rock mass classification systems of Laubscher (1990) and Bieniawski (1989) as well as the Q system after Barton et al. (1974). The summary given in Table 4-1 compares the rock mass condition of the sites based on the Geological Strength Index (GSI). GSI has been calculated as the RMR value (Bieniawski, 1989) less 5. Note that when stress induced extension fracturing is included in the rock mass classification, GSI values are significantly reduced. For full details reference can be made to the Appendices A1 through A5.

Support Capacity

Mapping of existing support systems was conducted concurrently with rock mass classification. An assessment of installed tendon support capacity was then conducted using the laboratory tested yield strength of the different support units present. This allows the different sites to be compared on the basis of the level of tendon support present before the application of shotcrete. In Table 4-1 the test sites can be compared based on installed support and the expected capacities of each support system.

Instrumentation

Table 4-2 gives a summary of the instrumentation installed at each of the test sites. Where possible a combination of instruments varying in complexity and sensitivity were used. Instruments used ranged from simplistic manual readout systems like laser targeting or video borehole scoping to the more complex digital systems, like the GMM’s, capable of continuous monitoring at a high resolution and accuracy. In addition a comprehensive photographic database was developed for each of the sites which proved to be instrumental in the analysis of the shotcrete - rock mass interaction.
**Shotcrete Application**

At the South Deep and Mponeng sites steel fibre reinforced shotcrete is used. The South Deep mix design includes the use of various admixtures and is sprayed according to a wet shotcrete process. At Mponeng no admixtures are used and application is with a dry spraying process. Shotcrete installations at Impala Platinum are un-reinforced and sprayed with a wet shotcrete process. Average spray thickness, EFNARC energy absorption, fibre content and UCS results are presented in Table 4-1. Further detail of the applied shotcrete mixes is available in the appendices A1 through A5.
### Table 4-1  Summary of test sites and installed instrumentation

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mining environment</strong></td>
<td>Gold - tabular de-stressing followed by mechanised drift and fill mining</td>
<td>Gold - tabular de-stressing followed by mechanised drift and fill mining</td>
<td>Gold – sequential grid</td>
<td>Gold – sequential grid with increased seismic risk</td>
<td>Platinum – scattered mining with in-stope crush pillars</td>
</tr>
<tr>
<td><strong>Site description</strong></td>
<td>Final pillars in a drift and fill trackless operation</td>
<td>Final pillars in a drift and fill trackless operation</td>
<td>Hangingwall crosscut at reef intersection</td>
<td>Footwall crosscut at a seismically active dyke intersection</td>
<td>Footwall drive 7m below the abutment of a haulage protection pillar</td>
</tr>
<tr>
<td><strong>Depth below surface</strong></td>
<td>2,550 m (de-stressed)</td>
<td>2,550 m (de-stressed)</td>
<td>3,037 m</td>
<td>3,245 m</td>
<td>1,200 m</td>
</tr>
<tr>
<td><strong>Loading Environment</strong></td>
<td>Quasi static loading at high stress with high stress changes and resulting damage expected</td>
<td>Quasi static loading at high stress with high stress changes and resulting damage expected</td>
<td>Reducing stress field due to over stoping operations</td>
<td>Dynamic loading, driven by seismicity</td>
<td>Quasi static loading at intermediate stress</td>
</tr>
<tr>
<td><strong>GSI (excluding ext. fracturing)</strong></td>
<td>60-65 (30-40 incl. fracturing)</td>
<td>65-70</td>
<td>Est. &gt;70 (42-49 incl. fracturing)</td>
<td>Unknown</td>
<td>Very high (&gt;75)</td>
</tr>
<tr>
<td><strong>Support type, capacity (excl. shotcrete)</strong></td>
<td>Swellex friction bolting, 1.4 x 1.4m spacing, 61 kPa</td>
<td>25mm rebar, 1.5m x 1.5m spacing, 100 - 120 kPa</td>
<td>107 kPa</td>
<td>25mm grouted rebar - forged head, 1.4m x 1.3m diamond pattern, 330 kPa</td>
<td>Split sets @ 1.4mx1.4m, WM&amp;L @ 2mx2m and 38 tonne anchors 2m apart, 214 kPa</td>
</tr>
<tr>
<td><strong>Applied Shotcrete</strong></td>
<td>SFR (wetcrete + add mixtures), &gt;50mm</td>
<td>SFR (wetcrete + add mixtures), 50-70 mm</td>
<td>SFR (drycrete), 70-120 mm</td>
<td>SFR (drycrete), &gt;50 mm est.</td>
<td>Unreinforced (wetcrete), &gt; 50mm</td>
</tr>
<tr>
<td><strong>Energy absorption</strong>&lt;br&gt; <strong>EFNARC (Joules)</strong></td>
<td>889 Joules</td>
<td>393 Joules</td>
<td>538 Joules</td>
<td>-</td>
<td>62 Joules</td>
</tr>
<tr>
<td><strong>Fibre density</strong>&lt;br&gt; <strong>(kg/m3)</strong></td>
<td>19 to 25 kg/m3</td>
<td>15 to 21 kg/m3</td>
<td>44 kg/m3</td>
<td>-</td>
<td>No fibre</td>
</tr>
<tr>
<td><strong>Compressive strength</strong>&lt;br&gt; <strong>(UCS @ age, type - size)</strong></td>
<td>28 MPa @ 30 days, Core - 68mm</td>
<td>30 MPa @ 30 days, Core - 100mm</td>
<td>36 Mpa @ +360 days, core - 100mm</td>
<td>23 MPa @ 136 days, Core - 65mm</td>
<td>-</td>
</tr>
<tr>
<td><strong>Flexural Strength (FS)</strong>&lt;br&gt; <strong>(FS @ age, standard - size)</strong></td>
<td>4.34 MPa @ 30 days, ASTM - 100x100x300mm</td>
<td>-</td>
<td>4.04 MPa @ 73 days, ASTM - 100x100x300mm</td>
<td>-</td>
<td>5.88 MPa @ 58 days, ASTM - 100x100x300mm</td>
</tr>
</tbody>
</table>

SFR = steel fibre reinforced  
WM&L = wire mesh and lace
### Table 4-2  Summary of installed instrumentation

<table>
<thead>
<tr>
<th>Mine</th>
<th>Name of site</th>
<th>Site detail</th>
<th>Displacement measurements</th>
<th>Strain / stress</th>
<th>Seismicity</th>
<th>Crack dilation (visual)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Laser targets</td>
<td>SPBX</td>
<td>MPBX</td>
<td>GMM</td>
</tr>
<tr>
<td>Gold Field's South Deep mine</td>
<td>Site 1</td>
<td>Ramp pillar</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>Site 2</td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pillar A</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pillar B</td>
<td></td>
<td>X</td>
<td>XX</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pillar D</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pillar E</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Anglogold Ashanti's Mponeng mine</td>
<td>Site 1 - 109 level</td>
<td>Eastern sidewall</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Western sidewall</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Site 2 - 116 level</td>
<td>Northern sidewall</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Southern sidewall</td>
<td></td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Impala Platinum's shaft #14</td>
<td>24 level</td>
<td>Station A</td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Station B</td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

#### 4.4 Analysis of results

Figure 4-12 shows combined monitoring and modelling results for Pillar B at South Deep site 2. Similar graphs exist for all of the other sites but not all of them are presented here. The Pillar B results have been instrumental in the analysis of the shotcrete-rock interface and the reader will be referred back to this figure frequently in the discussions that follow.

The graph plots measured sidewall displacement from each of the installed instruments on the LHS axis. Modelled site stresses and measured PPV’s are read off the RHS axis. Changes in deformation are seen to correspond very well with known changes in the mining sequence and modelled pillar stresses. Two important events are indicated - the increase in deformation rate on 4 August 2007 and the instantaneous jump in deformation on 6 Jan 2008.
4.4.1 Measured sidewall deformations

Figure 4-13 compares the different sites based on the sidewall deformation measured over the life of monitoring. Reported deformations have been measured using the instruments indicated in parenthesis at the specified depth of anchorage into the sidewall. Deformations at the South Deep pillar sites (read off the LHS axis) are orders of magnitude higher than deformations at the Mponeng and Impala tunnel sites (read off the RHS axis). This is expected given that the South Deep sites consist of final safety pillars that are heavily loaded. The pillar sidewalls are also much larger in size and far less supported than the sidewalls of the smaller tunnel sites. Shotcrete damage was noted to be substantially more severe at the South Deep sites making these sites ideal for the investigation of the mechanisms of shotcrete failure.
4.4.2 Modelled site stresses

Stress changes over the life of each site were assessed using the Map3D numerical modelling code.

Figure 4-14 compares stresses at the pillar sites (Pillars A, B, D and E of South Deep site 2) based on average pillar stress (APS). Modelled APS results show various dates of sudden change in the rate of stress increase (marked with arrows). These dates correlate well with known changes in the mining of nearby drifts and benches. Corresponding changes in measured pillar deformation rates were found at these dates. Pillar B, being the smallest of the pillars and being the closest to the mined working places, has undergone the most significant stress changes.

Figure 4-15 compares stresses at the various tunnel sites at Mponeng and Impala platinum based on the induced sidewall tangential stress as calculated by the Kirsch equations for stresses around a circular opening (refer to analytic solutions in elasticity – Ryder and Jager (2002)).
Significant de-stressing is noted at Mponeng 109 level. Movement of the rails onto which the laser range finder was mounted was captured by the laser target displacement measurements. Actual sidewall deformations were significantly less than the induced rail movements indicating that the applied shotcrete performed effectively. The application of 70 mm to 120 mm steel fibre reinforced shotcrete (44 kg/m$^3$) effectively stopped scaling of the sidewalls (which was noted to be quite prominent before spraying) whilst the low level of resulting shotcrete damage suggests that sidewall deformations were kept to the minimum during the de-stressing process.

Modelled stress changes at the Mponeng 116 level site are relatively low in comparison to the other sites. Damage to installed shotcrete at this site is attributed mainly to seismicity. Results from this site have been used to further our understanding of the effect of seismicity on shotcrete.

The Impala site, being shallower and situated in a lower virgin stress field, undergoes the highest increase in stress (16%) of all the tunnel sites. Site observations show that the unreinforced shotcrete remains undamaged throughout the test duration and that sidewall deformations are kept to the absolute minimum. As with the Mponeng 109 test site spalling was noted to be a considerable problem prior to the application of shotcrete. This site is believed to bear testament to the effectiveness of un-reinforced shotcrete at controlling spalling ground conditions at intermediate mining depths where moderate stress changes are expected.
Figure 4-14  Modelled stress changes for pillar sites (South Deep site 2)

Figure 4-15  Modelled stress changes for tunnel sites (Mponeng and Impala)
4.4.3 Strong ground motion and its influence on shotcrete

Applied shotcrete can be exposed to strong ground motions from nearby blasting activity or from large or nearby seismic events. Peak particle velocity or PPV, measured in metres per second, is a typical measure of the intensity of strong ground motions. PPV was measured at both the South Deep and the Mponeng 116 test sites using electronic peak velocity detectors (PVD's). Where possible the mines’ seismic databases were used to complement results obtained.

Drift and bench blasting at South Deep

Blasting operations at the South Deep trackless sections have been found to influence measured sidewall deformations and the resulting shotcrete damage. This is especially true after drift blasting that is situated very close to the pillars being monitored or after bench blasts which use a significantly higher explosives charge. Each charge in a typical “long-hole” bench blast is about 13.5 kg. This is significantly larger than a single charge in a typical stoping or development end on most South African mines.

It has been noted at the South Deep sites that blast events (not recorded seismic events) account for almost all of the measured PPVs. PPVs from recorded seismic events (when captured) were significantly lower in value. It is also noted that it is blasting (and not seismicity) that results in almost all of the recorded “instantaneous jumps” in sidewall deformation.

Figure 4-16 plots the measured GMM sidewall deformation of Pillar B (site2) along with recorded peak particle velocity (PPV). The plot begins in July 2007 at the start of drifting of an adjacent pillar. This drifting is located about 10 to 20 metres away from the pillar being monitored and although drift charges are smaller than bench charges, they still constitute significant PPVs. The following is observed from the figure:

- A series of recorded PPVs is indicated by the brown dots. Many of these are known drift blasts and can be seen to occur every one to two days. Many of these blasts are also noted to result in quite significant “instantaneous jumps” in deformation of 2 mm or more.
- The figure indicates two recorded seismic events. The first, a $M_L 0.3$ event, appears to correspond with an instantaneous jump in deformation on 8 August 2007 but closer inspection reveals that the two events are 13 hours apart and are not related. The second event, a $M_L 0.8$ on 14 August 2007, locates the closest to the test site (126 m away) and represents the highest seismically driven PPV experienced by the site over...
the life of monitoring. Neither of these events resulted in any measurable deformation change.

- The maximum recorded PPV for the blast events is about 70 mm/s which is three times larger than that recorded for the worst case seismic event (about 25 mm/s). Strong ground motions from the drift blasts are clearly driving the instantaneous deformations recorded at this site.

- Some blasts result in more deformation than others. Blasts having fewer PPV recordings (fewer brown dots) tend to induce less instantaneous deformation. It is suggested that the vibrational intensity of a dynamic event like a blast is one of the factors that will determine the scale of the resulting jump in deformation. Other obvious factors are the location and the charge mass of the explosive used to create the event.

**Figure 4-16** Comparison of induced deformations by seismic and blast type events

Sidewall deformations after each blast closely mimic measurements of continuous stope closure after blasting as reported by SIMRAC in 2003. Figure 4-17 illustrates this by comparing typical continuous stope closure after blasting (Malan, 2003) to continuous sidewall deformation after
blasting as recorded at South Deep. The same phases of post blast deformation are identifiable in the South Deep GMM recordings.

![Diagram of closure and time](image)

**Figure 4-17** Comparing continuous stope closure after blasting after Malan (2003)(LHS) to continuous sidewall deformations measured at South Deep (RHS)

The largest recorded instantaneous jump in deformation was 14 mm and was recorded by SPBX on Pillar B on 6 January 2008 (refer back to Figure 4-12). This jump corresponds with bench blasting of the same area that was drifted between July and December 2007. The blast resulted in significant damage to pillar B and its instrumentation.

Figure 4-18 shows the recorded PPVs for this event. Detonation of each individual explosive charge can be seen to occur at a delay of about 160 milliseconds. Individual charges registered measured PPVs of 200 mm/s and less. To validate, theoretical PPVs were calculated according to Equation 1 (Rorke, 1992). The equation provides an estimation of the resulting PPV (in mm/s) a distance of $R$ metres away from the detonation of a charge mass of $Q$ kg of Anfex explosive.

$$PPV = 1143 \left( \frac{R}{Q^{0.5}} \right)^{-1.6}$$  \[Rorke, 1992\]  

Assuming a 13.5 kg charge mass of emulsion explosive, having a relative weight strength of 0.83, the theoretical PPV 10 to 20 metres away from an equivalent Anfex charge are found to be between 70 and 210 mm/s. This agrees remarkably well with the measured values.

Two anomalously high PPVs of almost 3 m/s are however noted quite late in the sequence of the blast. As the movement responsible for these two recordings exceeds the travel capacity of
the PVD instrument, it is believed that these values are high but not accurate. Their existence is however of great importance. It is believed that these recordings are due to a sudden site response as rock or support failure occurs. These PPVs are thus amplified by the site's response to continued PPV exposure and are not driven by the initial blast energy directly. These events would correspond with the moment of instantaneous jump in deformation introduced earlier. It is interesting to note that several explosive charges (29 separate charges) detonated before this failure occurs. Failure is thus noted to be the product of prolonged exposure to a succession of smaller strong ground motions. As suggested earlier, the vibrational intensity of an event plays a role in the degree of instantaneous deformation induced and thus the damage that can be expected.

![Chart showing recorded PPVs for the 6 January 2008 bench blast](image)

**Figure 4-18  Recorded PPVs for the 6 January 2008 bench blast**

**Seismicity at Mponeng 116 level**

It has been suggested that vibrational intensity is one of the factors governing the scale of induced deformation. PVD and seismic data from the Mponeng 116 site were analysed in an attempt to relate the magnitude of PPV to recorded sidewall deformation.
Blasting at the Mponeng 116 site is of the form of typical narrow-tabular reef stoping. Individual explosive charges are smaller and are situated much further away from the monitoring site meaning that damage from blasting was not expected. Instantaneous deformations at this site were driven by the seismicity in the area.

Figure 4-19 presents GMM deformations measured on the southern sidewall of the Mponeng 116 level site. Deformation is limited to 1.4 mm and almost all of this is induced in a succession of instantaneous jumps that correspond with a series of recorded seismic events. Measured PPVs from the installed PVD are also plotted and can be read off the RHS axis.

**Figure 4-19 Seismically induced deformations at Mponeng 116 level**

The first thing to notice is that induced jumps in deformation are significantly lower than they were at South Deep. A number of possible reasons are suggested. Firstly the 116 level tunnel site is a smaller and better supported excavation and will thus be more resilient to damage. Secondly the pillar sidewalls at South Deep are more damaged and are exposed to high quasi-static deformations driving continued deterioration. Thirdly it is likely that the South Deep blast events, taking place over a longer duration of time, are vibrationally more intense than the large seismic events at Mponeng which dissipated quite quickly.
The second thing to notice is that the PPV recordings, in what can only be described as PVD instrument error, fail to correlate with any of the major seismic events indicated on the figure. Recorded PPVs appear rather high, are random and seem unrelated to any known driving processes. Theoretical PPVs were thus calculated based on published far field PPV relations from McGarr, Spottiswoode and Hedley. A similar relation provided by ISS International (who oversee and run the seismic system at Mponeng) was also considered. The relations are shown in equations 2, 3, 4 and 5.

\[
\log(PPV) = 0.758(M_L) - 1.528(\log R) + 3.375 \quad [\text{ISSI}] \tag{2}
\]

\[
\log(PPV \times R) = 0.57(M_L) + 1.95 \quad [\text{McGarr, 1981}] \tag{3}
\]

\[
\log(PPV \times R) = 0.5(M_L) + 2.81 \quad [\text{Spottiswoode, 1984}] \tag{4}
\]

\[
PPV = 4000 \left( \frac{R}{M_L^{1.6}} \right) \quad [\text{Hedley, 1990}] \tag{5}
\]

Where PPV is the peak particle velocity in mm/s, \( M_L \) is the local magnitude of seismic event and \( R \) is the distance of the point of reference to the seismic source in metres.

Table 4-3 details the calculated far field PPV estimates for all recorded events above \( M_L \) 1.0. The measured instantaneous jumps in GMM deformation are also listed.

Figure 4-20 shows the relationship between calculated PPV and measured jump in GMM deformation. PPV is calculated as being the average of the four equations presented. Of all the seismic events over the life of the site only 3 result in measurable jumps in deformation. These three events are the only events having calculated PPVs higher than 35 mm/s. A "no deformation" PPV threshold appears to exist below which no measurable jump in deformation occurs. It is expected that this PPV threshold will be site specific depending on the prevailing support, rockmass and stress conditions. It also appears that the higher a calculated PPV is above this threshold, the higher the induced jump in deformation will be. There are however currently insufficient data points to determine a more specific relationship.
### Table 4-3  Calculated PPV results and measured instantaneous deformations

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<th>Distance to site “R” (m)</th>
<th>* Measured PPV (mm/s)</th>
<th>Calculated PPV (mm/s)</th>
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<th>Measured deformation (mm)</th>
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*Note: Measured PPV values should not be trusted as instrument malfunction is suspected*

---

### Figure 4-20 Relationship between PPV and induced deformation
Conclusion

Larger charge mass blast events are capable of inducing significant jumps in site deformations. Site deformations after these blasts closely resemble continuous closure in tabular stopes as measured by Malan (2003). Continued exposure to such events can lead to damage to the installed shotcrete and eventual failure of the site. The vibrational intensity of a strong ground motion (as well as the duration of time over which it occurs) appears to influence the amount of deformation induced and the likely resulting damage.

In a case where strong ground motions are seismically driven the amount of instantaneous deformation caused seems dependant on the PPV value. Only events producing PPVs above a site specific “no-deformation” PPV threshold are expected to result in deformations and damage.

4.4.4 The depth of fracturing and its influence on shotcrete

Multipoint borehole extensometers (MPBX) were installed at both the South Deep sites to assess the distribution of fracturing and dilation within the pillar sidewalls. Figure 4-21 shows the MPBX results for instruments installed at South Deep site 1 as well as South Deep site 2, Pillar D. The graphs plot the total measured displacement with time at different anchor points.

At both sites the measured deformations increase consistently with increased depth of anchorage. Deformations measured between the shallowest and the deepest anchors are equal to or greater than the deformations measured between the borehole collar and the first anchor point. The physical interpretation of this is that sidewall extension fracturing must be extending deep into the sidewall, well within the zone between anchor points. Borehole camera work conducted at site 1 showed fracturing at a depth of 2.5 m so severe that dislocation of the hole had occurred. Borehole camera work from site 2 Pillar D however delivered surprising results with intensive fracturing clearly extending no further than 1.5 m into the sidewall. The borehole camera work failed in this case to support the MPBX readings which indicates that the intensity of fracturing is probably locally variable and the behaviour is more complex than expected. Generally it can be accepted that under the South Deep pillar conditions the depth of fracturing is likely to extend deeper than 2.5 m.
Figure 4-21  **MPBX results for (a) SD site 1 and (b) SD site 2 Pillar D**

Figure 4-22 shows the MPBX results for site 2 Pillar B. This pillar is of particular interest to the project given the high deformations and damage recorded here. Readings stop abruptly in early January 2008 following severe damage to the pillar and instruments caused by a nearby bench blast.

The following important observations are made:

- Deformations are the greatest between the collar of the hole and the 1.8 m anchor depth. This is expected as the most severe fracturing and dilation is known from the borehole camera work to occur in the first metre of sidewall depth.
• Between 1.8 m and 2.8 m depth an appreciable amount of deformation occurs. Borehole camera record from nearby Pillar A (similar in size) shows distinct fracturing extending as deep as 2.5 m into the pillar sidewall. Safety concerns at the deteriorating conditions at pillar B resulted in no borehole video being taken here. It is however fair to assume that fracturing extends deeper than 2.5 m given the higher stresses acting here.

• Between 2.8 m and 3.8 m much less deformation occurs suggesting that fracturing and dilation drops off significantly past 2.8 m depth. The results are actually surprising showing less displacement at 3.8 m depth than at 2.8 m depth.

• On 4 August 2007 a sudden increase in deformation is noted on all the anchor points. This date was first highlighted in Figure 4-12 due to the great importance that it holds in the analysis that follows. In section 4.4.5 it will be shown that this date corresponds with the joining of individual cracks within the applied shotcrete to form a loose slab.

A detailed discussion on the mechanisms and processes at play on this date is included in the sections that follow. For now it is sufficient to note that a loose slab of shotcrete has formed close to the installed MPBX and that the magnitude of the measured increase in deformation is similar for all three of the MPBX anchor points. This means that most of this deformation has occurred in the first 1.8 m of sidewall depth.

Figure 4-23 shows the sidewall of South Deep pillar B.
The relative positions of installed tendons, the newly developed shotcrete slab and the MPBX instrument are shown. Section A-A is defined and is then schematically represented in Figure 4-24. Tendon support consists of 2.4 m long resin rebar spaced 3 m apart either side of the shotcrete slab. The resin rebar zones of influence are represented by the two isosceles triangles. The installed MPBX is located about 0.6 m from the edge of the slab. MPBX results show that all the recorded instantaneous deformation (at the time at which the slab first forms) occurs within the first 1.8 m. It appears that this deformation is focused in the area falling outside of the zone of influence of the installed tendons as indicated in the Figure 4-24. The shotcrete was therefore resisting deformation and preventing unravelling of broken rock outside the zone of influence of the tendon support.
To better understand the failure of shotcrete and its interaction with the rockmass to which it is applied, it was necessary to monitor and observe the processes at play whilst the shotcrete is
being extensively deformed. South Deep site 2 was an ideal test site owing to the high sidewall
deformations recorded and the resulting high levels of shotcrete damage that occurred.
Observations of failure at the other sites have been used to support the arguments made.

Two distinct stages of shotcrete failure have been identified through detailed mapping of
shotcrete crack propagation.

- In the primary stage, isolated primary cracks form at random positions on a monitored
  face. Primary cracks are independent of one another and the performance of shotcrete
  is not believed to be significantly affected during this stage.

- Once primary cracking has propagated far enough, shotcrete failure enters a secondary
  stage. Secondary failure involves the joining or interaction of primary cracks after which
  sidewall deformations are noted to increase significantly. The onset of the secondary
  stage of failure is believed to be a good indicator that the support capacity of applied
  shotcrete has been compromised.

The observations leading to the discovery of these stages are discussed under the relative site
headings below.

**Observations made at South Deep**

Figure 4-25 presents the progression of shotcrete cracking on Pillar B at South Deep site 2 as at
the beginning of August 2007. Before this date, five primary cracks (numbered 1 to 5) were
propagating along separate failure paths. Failure of the applied shotcrete is in the primary
stage. Somewhere between monitoring dates 29 July and 9 August 2007 cracks 2 and 3 join
creating the isolated and loose slab which is noted to be pushed outwards as pillar loading
continues. Failure of the shotcrete on Pillar B has now entered the secondary stage of failure.
The effect of this event on the stability of the site is demonstrated by Figure 4-26. The figure shows a plot of cumulative crack length with time that has been superimposed onto the GMM displacement readings from Figure 4-12.

An initial increase in the rate of deformation correlates with the start of mining of an adjacent pillar (drift 4). This is soon followed by a second sudden increase in deformation rate following a typical drift blast on 4 August 2007. Instantaneous blast deformations are suddenly amplified and the overall deformation rate increases significantly. As far as can be determined this date does not correspond with any new mining and it is believed to mark the formation of the loose slab and the start of the secondary stage of shotcrete failure.

During the primary stage (between the start of the mining of drift 4 and the formation of the loose slab), the rate of measured deformation remains constant at 0.22 mm/day. The existing primary cracks are also noted to be propagating at a semi-constant rate of about 3.3 cm/day.
The secondary stage starts with the joining of primary cracks and the formation of the loose slab. The measured rate of sidewall deformation increases five fold from 0.22 mm per day to 1.16 mm per day. No other changes have been introduced with the only mining taking place still being that of the adjacent drift 4. The ability of the support system to resist imposed sidewall deformations has been significantly reduced – so much so that 5 times the amount of deformation is now occurring under the same loading conditions. The secondary stage of shotcrete failure is thus joined by a considerable decrease in the performance of the applied shotcrete.

The rate of crack propagation is also noted to drop off significantly from 3.3 cm/day to 0.6 cm/day after slab formation. The mechanism presented in Figure 4-24 of section 4.4.4 has come into play with bulking and lateral sidewall deformations now being focused onto the dilation of the crushed rock directly behind the newly formed slab instead of onto the further development of existing cracks.

Based on the evidence from this site it appears that the formation and propagation of primary cracks has little to no influence on the performance of shotcrete. It is only after entering its secondary stage that shotcrete failure becomes a serious concern.

**Figure 4-26  Correlation between crack propagation and sidewall deformation**

The secondary stage starts with the joining of primary cracks and the formation of the loose slab. The measured rate of sidewall deformation increases five fold from 0.22 mm per day to 1.16 mm per day. No other changes have been introduced with the only mining taking place still being that of the adjacent drift 4. The ability of the support system to resist imposed sidewall deformations has been significantly reduced – so much so that 5 times the amount of deformation is now occurring under the same loading conditions. The secondary stage of shotcrete failure is thus joined by a considerable decrease in the performance of the applied shotcrete.

The rate of crack propagation is also noted to drop off significantly from 3.3 cm/day to 0.6 cm/day after slab formation. The mechanism presented in Figure 4-24 of section 4.4.4 has come into play with bulking and lateral sidewall deformations now being focused onto the dilation of the crushed rock directly behind the newly formed slab instead of onto the further development of existing cracks.

Based on the evidence from this site it appears that the formation and propagation of primary cracks has little to no influence on the performance of shotcrete. It is only after entering its secondary stage that shotcrete failure becomes a serious concern.
Supporting observations from Mponeng 116 level

Figure 4-27 shows a series of primary cracks on the southern sidewall of the Mponeng 116 level site. Crack 2 is one of the first cracks to form and is located directly at the position of GMM installation. Two months later, crack 3 forms very close to the existing crack 2 about half a metre below the installed GMM. The formation and development of crack 3 results in no measurable changes in deformation at crack 2 as recorded by the GMM. This shows that the influence of a single crack is rather localised and that individual primary cracks are independent of each other even when closely spaced. This observation supports the suggestion that the development of primary cracks has a limited influence on induced site deformations.

![Observed primary crack propagation at the Mponeng 116 site](image)

**Figure 4-27  Observed primary crack propagation at the Mponeng 116 site**

4.4.6 Observed modes of shotcrete failure

Barrett and McCreath (1995) studied the mechanisms of shotcrete failure under static loading conditions in shallow to intermediate depth environments. They suggested that shotcrete can fail in the six failure modes listed below and illustrated in Figure 4-28.
1. Adhesive failure
2. Direct shear failure
3. Compressive failure
4. Flexural failure
5. Punching shear failure
6. Pure tensile failure

Failure according to either the flexural or the punching shear mode is subject to the loss of adhesion between the shotcrete and the rock. The propagation of a crack under the flexural failure mechanism is illustrated in Figure 4-29.

![Figure 4-28 Six potential modes of shotcrete failure after Barrett and McCreath (1995)](image-url)
In deep level environments the immediate skin of excavations is usually fractured and zones of crushed rock were frequently observed behind shotcrete applications. As illustrated in Figure 4-30 loss of adhesion is almost certain to occur via separation of one form or the other quite early in the life of an underground site under these conditions. With loss of adhesion confirmed, the flexural and the punching shear mechanisms become kinematically possible, especially during the secondary stage of shotcrete failure.

Figure 4-30  Loss of adhesion due to slabbing and crushing of the rock
In keeping with the theme of primary and secondary stages of failure the mechanisms involved in the formation of each stage have been investigated. A common observed example of the interaction of primary and secondary cracking is shown in Figure 4-31.

![Primary and secondary cracking on pillar D](image)

**Figure 4-31  Primary and secondary cracking on pillar D**

**The nature of the primary stage of failure**

The perpendicular orientation of failed fibres across dilated primary cracks suggests failure in tension (Figure 4-32). It can however not be conclusively said whether the actual mode of failure during the primary stage is flexural or in pure tension. In both cases failure occurs in tension but the driving force behind development of the cracks is vastly different. The problem when analysing cracks in the primary stage of development is that there is no practical way of observing the details behind the formation of the cracks. These are hidden inside the shotcrete and only the surface trace of the crack is observed. During the secondary stage of failure enough crack interaction has occurred allowing a better view of the forming cracks. This is discussed under the next heading.
There is evidence supporting both the flexural and the pure tension modes of failure. Expected loss of adhesion and distributed loads from the dilation of the observed crushed material tends to support the flexural mode. There is also strong evidence discussed under the next heading that supports a flexural mode of failure during the formation of secondary cracking. Failure in the flexural mode is expected to be driven by localised bulging effects. On the other hand, the fact that primary cracks were observed to extend over large distances and far past prominent tendon anchor points (refer back to Figure 4-25) suggests failure in pure tension. In this case the failure is believed to be driven by processes that are more deeply set in the rockmass and that are acting over a much larger area. It is likely that the primary stage of failure occurs as a combination of both these modes.

![Figure 4-32 Close-up view of a primary crack on pillar A. Note the fibre orientation.](image)

**The nature of the secondary stage of failure**

Figure 4-33 shows photographs of a secondary crack that is in the process of formation. In the LHS photograph one is looking onto the sidewall. The horizontal trace of the secondary crack can be seen to the right. The initial primary crack can clearly be seen extending vertically down the centre of the photograph. In the RHS photograph one is looking parallel to the face onto the surface of the primary crack (note that the LHS slab has been forced downwards allowing the
picture to be taken). The photograph shows a clear section through the forming secondary crack which is seen to be developing from right to left under tension. The photograph bears an indisputable resemblance to the flexural failure mechanism of Figure 4-29.

Figure 4-33  Front view (LHS) and side view (RHS) of typical secondary flexural failure

Figure 4-34 shows another commonly observed example of secondary failure along an inclined failure plane. The failure resembles the punching shear mechanism as described by Barrett and McCreath (1995). The punching shear mechanism relies on the development of high shear forces within the shotcrete which are typically believed to occur at installed support tendons. The observed case is however different. Being nowhere near to any installed tendons, other mechanisms, capable of inducing the forces required, must be at play. A probable mechanism is presented below and is illustrated in Figure 4-35.

It has been noticed that these inclined plane failures tend to occur where the sidewall profile is very undulating and that the failure usually occurs at the edge of the bulged or protruding part of the undulation. Crushed rock usually noted behind the bulged area places load on the lower part of the applied shotcrete. As the crushed rock particles are squeezed outwards the shotcrete slab cantilevers placing stress at the top of the bulge which is (in relative terms) fixed due to better shotcrete-rock adhesion. Applied shotcrete is often thinner at rock undulations adding to the likelihood of failure here. A tensile crack forms perpendicular to induced movements which creates a cracked surface that is orientated along a shallowly inclined plane.
Figure 4-34  Secondary cracking following the punching shear mechanism (pillar D)
4.5 Discussion on the interaction of shotcrete and the rockmass

The findings of the previous sections can be summarised as follows:

- Strong ground motions from seismicity or large blast charge masses do result in significant jumps in sidewall deformation and corresponding crack propagations. Nearby events, especially those of a higher vibrational intensity can be particularly strong inducers of damage.

- In-depth analysis of crack propagation identified that shotcrete fails in two distinct stages.
  - The primary stage of failure is identified by the formation and propagation of individual "primary" cracks throughout the shotcrete installation. This stage of
failure is not believed to result in a noticeable drop in the performance of installed shotcrete.

- The secondary stage follows once primary cracks have propagated far enough to join or interact. In many cases the joining of primary cracks is achieved through the development of secondary cracking. This stage of shotcrete failure is synonymous with a marked drop in the performance of the applied shotcrete and is accompanied by significant increases in sidewall deformation and damage.

- Analysis of MPBX deformations shows that when secondary failure occurred, most deformation occurred within first 1.8 m and between tendons. This indicates that the shotcrete was probably containing the broken rock outside the zone of influence of the tendon support.

### 4.5.1 The failure of shotcrete under quasi-static loading

Figure 4-36 illustrates quasi-static loading of a typical rockmass at depth. The rockmass could represent a pillar as at South deep or a tunnel sidewall as at any of the other sites. The near zone is defined as the area that exists in the immediate sidewall falling within the typical working length of installed tendon support. The far zone refers to deeper areas outside of the working tendon length. Far zone and near zone pressures “Pf” and “Pn” are also indicated.
The function of applied shotcrete is to preserve the fabric of the rock mass by acting as a membrane through which loads can be transferred. In doing so, shotcrete effectively prevents the movement of key blocks and the unravelling of the rockmass.

As the rockmass is loaded deformations are induced by dilation of the fracturing that exists both in the near and far zones. The dilating surface continues to load the shotcrete lining resulting in the development of primary cracks in tension according to either the flexural or pure tension modes of failure.

Pure tensile loading, is due to displacement that takes place beyond the zone supported by tendons, where Pf is the dominant driver of deformations. The entire zone supported by tendons deforms as a large fractured rock beam. The outer surface of the fractured rock beam, which includes the entire thickness of the shotcrete layer, is in pure tension and this could cause or contribute to the development of primary fractures. The shotcrete layer is effectively elongated over the height of the sidewall or width of the roof as shown in Figure 4-37. This elongation causes the tensile strain, which is determined as follows:

**Figure 4-36  The mechanism of primary shotcrete failure on a loaded pillar**

The function of applied shotcrete is to preserve the fabric of the rock mass by acting as a membrane through which loads can be transferred. In doing so, shotcrete effectively prevents the movement of key blocks and the unravelling of the rockmass.

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\[ a' = \sqrt{a^2 + \delta_f^2} \]

\[ \sigma_t = \frac{E(\sqrt{a^2 + \delta_f^2} - a)}{a} \]

where:

\( E \) is the shotcrete elastic modulus (typically 30 GPa)

\( a \) and \( a' \) are the initial and final lengths of the shotcrete, and

\( \delta_f \) is the far displacement

**Figure 4-37: Elongation of shotcrete due to far displacement**

These tensile loads are likely to be very small and will only contribute to the total loading.

Where \( P_n \) is more dominant (as is usually the case during the secondary stage of shotcrete failure) the flexural mode of failure is expected. It is believed that a combination of both these modes is responsible for damage during the primary stage of shotcrete failure. The performance of the shotcrete is not expected to be greatly affected during this, the primary stage of failure.

Once individual primary cracks have propagated far enough, they begin to interact and may even join. This is often accompanied by the development of secondary crack systems. When primary and secondary cracks combine, forming free shotcrete slabs, the structural integrity of
the support lining is compromised. Smaller blocks within the crushed substrate are now able to move more freely and the crushed rock can now move outwards resulting in localised failure which soon spreads out to the rest of the supported area. Shotcrete has now entered the secondary stage of failure and increased sidewall deformations and damage can be expected.

Secondary cracks during the secondary stage of failure have, depending on the conditions present, been noted to develop according to either the flexural or the punching shear modes of failure as indicated in Figure 4-38.

Figure 4-38  Mechanisms of secondary shotcrete failure

4.5.2 Suggestions for prolonging the life of shotcrete installations

The function of shotcrete in tunnel support is to preserve the fabric of the rock mass, allowing arching to develop in the rock mass itself as stress redistribution occurs (Barrett and McCreath,
From the evidence presented here, the support capacity of applied shotcrete is significantly reduced after it enters the secondary stage of failure. Shotcrete design needs to focus on preventing or delaying the onset of failure of the secondary stage. To prevent the joining or interaction of primary cracks one of two things needs to be done:

- Prevent the joining of primary cracks by forcing an engineered path of crack propagation. The concept of engineering the path of failure is used in the building industry when joints are cut into walls and floors to control the location of concrete cracking due to initial shrinkage effects and later earth movements.

OR

- Slow down the formation and propagation of these primary cracks. This needs to be controlled by a well defined support design process that finds an appropriate balance between the thickness of application and the strength and type of the shotcrete used. It is here that a full understanding of the mechanisms in which these cracks are formed and propagate is important.

### 4.6 Summary and conclusions

Five test sites were identified, established and instrumented at three different South African mines.

- At the two South Deep sites quasi-static pillar loading lead to high deformations of 70 mm and more over the 14 months of testing. Site 1 served as a trial run during which numerous lessons were learnt and improvements were made in preparation for the development of site 2. The South Deep sites were ideal for the analysis of shotcrete failure mechanisms and the shotcrete-rock interface because of the high levels of shotcrete damage that was occurring. The sites were also exposed to strong ground motions from both seismicity and nearby bench blasts.

- Instrumentation and monitoring at the Mponeng 109 site had to be downscaled due to unexpected changes in the mining strategy. The site captures the performance of fibre reinforced shotcrete at deep level where over-stoping or de-stressing is taking place.

- The Mponeng 116 test site, originally not part of the testing programme, was established to investigate the influence of strong ground motions from seismicity on the performance of shotcrete. The site was also instrumental in observing shotcrete in its early stages of
failure and helped to confirm findings from the analysis of failure mechanisms from the South Deep sites.

- The Impala Platinum site captures the effectiveness of un-reinforced shotcrete in controlling spalling ground conditions at intermediate mining depths where moderate stress changes are expected.

Thorough site assessments have led to the following conclusions and improved understanding.

- At intermediate depth a 50 mm thick layer of un-reinforced shotcrete has been shown (by the Impala test site) to effectively control tunnel deterioration where sidewall spalling occurs, but the stress change is limited.

- In deep level mining, 70 to 120 mm steel fibre reinforced shotcrete (44 kg/m³) has been effective in stopping severe sidewall spalling whilst effectively maintaining the integrity of the tunnel during subsequent de-stressing (Mponeng 109 level). Steel fibre reinforced shotcrete has also been noted to be more resistant to corrosion than wire mesh and lacing.

- Analysis of deformations due to large seismic events at Mponeng 116 level suggests that induced deformation (and damage to shotcrete) is related to the PPV on the excavation surface resulting from the strong ground motion. Damage is however only expected with PPV’s greater than a site specific threshold value.

- Instantaneous jumps in sidewall deformation measured at the South Deep sites at the time of nearby blast events show that blasting activity can be a particularly strong inducer of damage. The vibrational intensity of a blast event has been found to affect the degree of deformation that results. At South Deep damage from seismicity in the area was virtually insignificant when compared to damage from nearby bench blasting.

- In-depth analysis of shotcrete crack formation and propagation at the South Deep sites identified that shotcrete fails in two distinct stages.
  - The primary stage of failure is identified by the formation and propagation of individual “primary” cracks throughout the shotcrete installation. This stage of failure is not believed to result in a noticeable drop in the performance of installed shotcrete.
  - The secondary stage follows once primary cracks have propagated far enough to join or interact. In many cases the joining of primary cracks is achieved through
the development of secondary cracking. This stage of shotcrete failure is synonymous with a marked drop in the performance of the applied shotcrete and is accompanied by significant increases in sidewall deformation and damage.

- The exact mechanism behind the formation of cracks during the primary stage has not been conclusively determined but it is known that cracks form in tension. It is further understood that the damage caused by these cracks and its influence on the overall shotcrete performance is localised and thus rather limited.

- Cracking during the secondary stage of shotcrete failure has been observed to commonly occur according to the flexural failure mechanism. Many examples of secondary cracking according to a mechanism resembling punching-shear have also been documented. In both cases the cracks still form in tension. Secondary cracking has a substantial influence on the overall performance of shotcrete.

- Analysis of MPBX deformations shows that when secondary failure occurred, most deformation occurred within first 1.8 m and between tendons. This indicates that the shotcrete was probably containing the broken rock outside the zone of influence of the tendon support.

4.7 Recommendations for further work

- There is scope for further study into a relationship between PPV and induced instantaneous deformation. The existence of a “no deformation” threshold needs to be confirmed and the relationship after this threshold needs to be quantified.

- In light of the instrument errors that have occurred, it is advisable that anyone looking to measure PPV’s consider finding a suitable upgrade to the original PVD design. The CSIR has recently developed a new instrument for the measurement of strong ground motions which is currently being tested and which will hopefully overcome the shortcomings of the earlier design.

- There is scope to further study the depth of fracturing in deep level, high deformation environments. Large variations have been noted to exist between depths of fracturing measured using different types of instruments. It is believed that the intensity of fracturing is probably locally variable and the behaviour is more complex than may be expected.
The concept of the primary and secondary stages of shotcrete failure deserves further testing. If possible, back analysis of underground failure incidents should be conducted, assessing the stage and degree of shotcrete failure at the time of site collapse.

- There is scope to develop a surface test under controlled laboratory conditions that can effectively demonstrate and repeatedly produce a progression from primary to secondary stage failure as defined in this research.

- If such a test can be devised then it would be useful to conduct a series of tests that investigate the effect of varying common design parameters like strength, thickness and type of reinforcement on the duration of the primary stage of failure as identified by this work. Can the shotcrete be engineered to effectively prolong the time before entering the secondary stage?
5 Use of numerical modelling to estimate shotcrete requirements using a ground reaction curve approach

5.1 Introduction

5.1.1 Objectives of numerical modelling

A programme of numerical modelling has been carried out to examine rock and support behaviour, taking account of the complexity of rock interaction with support systems.

An understanding of the mechanisms of rock mass deformation around a tunnel is fundamental to the selection of an appropriate support system. The deformation magnitudes and failure mechanisms that occur as a result of the changes in field stress loading on a mine tunnel are complex, being strongly influenced by local geological conditions, and it is difficult to develop generalised ground responses based on a limited number of underground observational sites. Consequently, it was decided to investigate the relationship between deformation, damage and support requirements for a large number of easily defined geotechnical conditions using numerical models, with specific focus on the effect of support pressure and role that shotcrete may play.

Broadly, shotcrete is assumed to serve several roles:

- **Structural support** – where it is sufficiently thick to provide support to an entire tunnel, as a full shell that resists deformation applied to it.

- **As fabric between tendons** – here the tendons provide the primary support capacity, anchoring potentially loose rock to stable, confined rock some depth into the walls of a tunnel. Shotcrete then provides a fabric between tendons in which it restrains a roughly wedge-shaped volume of ground whose size is a function of tendon spacing.

- **Prevention of spalling or strain-bursting in a tunnel**, near the face prior to application of tendon (or permanent) support.

Numerical modelling was used with the objective of assessing the capacities in all three circumstances. One of the problems has been that shotcrete can be applied in a wide range of thicknesses and strengths, and hence the modelling has focused on determining the expected displacements (in terms of Ground Reaction Curves) for a range in geotechnical environments.
Different analytical methods can be used to assess the shotcrete design required to deliver specific support capacities. With this in mind, the objectives of the modelling programme were to:

- Derive the magnitude of anticipated rock mass movements around tunnels under a range of typical rock mass and stress conditions found in South African mines.
- Interpret these movements in terms of the deformations that will be applied to a layer of shotcrete – both across an entire tunnel wall, and between bolts.
- Derive Ground Reaction Curves (GRC) relating deformation to applied support pressure in each case.
- Assess the effect of changes in stress and excavation size on these deformation-support pressure relationships.
- Assess the typical distribution of loads induced in a shotcrete layer and broadly assess how these are influenced by bolting, bond strength (debonding) and fragmentation of the rock mass behind the shotcrete.

### 5.1.2 Workshop team

To determine the nature of numerical modelling required to assess shotcrete behaviour under the range of conditions encountered in South African mines, and to determine the objectives of the modelling programme outlined above, a series of workshops was held. The participants included:

- William Joughin, SRK
- Tony Leach, Itasca Africa
- Alan Naismith, SRK
- Thandle Dlokweni Itasca Africa/SRK
- Jody Thompson, SRK
- Julian Venter, SRK
- J Dube, SRK
- Kevin Le Bron, Independent Rock Engineer
- Dr Graham Howell, SRK
Dave Ortlepp, SRK

Several different approaches to represent the loading applied to a shotcrete support system were proposed and investigated. In addition to data from the underground sites (Chapter 4), a brief literature survey was conducted to investigate whether any previous SIMRAC project data could be applied to assist in model calibration. Through this exercise, it was proposed to apply a methodology to determine the unstable depth between rock bolts to obtain the deadweight and dynamic loading support requirements. Although there is merit in this approach, the project team felt that predicting the maximum deformation that the sidewalls and hangingwall would undergo would be more useful.

5.1.3 Numerical modelling project approach

All two dimensional models were set up with the universal distinct element code, UDEC. This code was selected for 2D analysis because UDEC is an inelastic, two-dimensional discrete element code that is able to represent the rock mass as discrete joint, or fracture, bound blocks with infinitely variable properties that can be assigned to any contacts between any blocks. In addition the blocks themselves can be permitted to deform either elastically or yield in response to assigned failure criteria. The model is therefore able to represent a rock mass that can break up and fragment under applied loading conditions – and reasonably represent the rock mass that may need to be contained by the shotcrete lining in a tunnel.

Initially a series of numerical model options (or numerical “laboratories”) was set up and compared to assess their ability to investigate shotcrete behaviour and requirements under a range of conditions. These are described below for completeness, including reasons for adoption of a final two-dimensional model geometry as a means of comparing geotechnical environments.

Initially, explicit modelling of the effect of shotcrete on the stability of the tunnels was conducted (actual sidewall deformation/closure recorded). The objective of this was to determine the effect that shotcrete would have on reducing the deformation around the tunnels. The results showed that shotcrete had a slight effect on the overall maximum deformation under the relatively high stress regimes encountered in South African gold and platinum mines, but due to the lack of confidence in the approach, the project team decided to follow a different methodology. The project team was not sure whether the effect of shotcrete on the sidewall was being accurately modelled using the application of the structural element in UDEC, through which a beam element with flexural and axial bending properties connects, and applied point loads, at zone...
grid points along the sidewalls and hangingwall. As a result, the explicit modelling of shotcrete was then excluded from the models.

It was then decided that the Ground Reaction Curve (GRC) concept was the more appropriate method to determine the shotcrete requirement (actual sidewall deformation/closure is recorded as internal support pressure is reduced). Applying this method, there are also two different approaches:

- Determining the ground reaction curve for the midpoint of the sidewall and hangingwall (this approach was followed using the Universal Distinct Element Code UDEC)
- Determining the ground reaction curve for the midpoint between two tendon support units.

It was decided to follow the first approach.

### 5.1.4 Model method and geometry selection

#### Choice of modelling code

In many instances in South African mines, shotcrete is used to support a rock mass which is already, or becomes, highly fragmented behind the applied lining. It was therefore considered important to use numerical models which account for the discontinuous nature of the rock mass. The code UDEC was selected due to its ability to explicitly represent planes that subdivide the rock mass and represent failure using industry-accepted constitutive models. The discontinuities and blocks created in the models are included in a geometry that is representative of typical damaged rock masses observed around underground tunnels. Large displacements along discontinuities and rotations between blocks are allowed, which is considered representative of the real inter-fragment movement in a damaged and un-ravelling rockmass on the skin of a tunnel. Individual blocks in UDEC can behave as either rigid or deformable materials that respond in accordance with a prescribed linear or non-linear stress/strain law.

#### Model geometry selection

To assess shotcrete support requirements and the range of possibilities for interaction with the rock mass, a series of numerical “laboratories” was set up using the two dimensional finite difference code UDEC. These were designed to examine various mechanisms.
Initially, three “laboratories” were created:

- A simple wedge that can be pushed (or ejected) through a shotcrete membrane, which can be bonded to the rock mass either side of the wedge (Figure 5-1 below).

- A “realistic” tunnel in bedded and jointed quartzite (See Figure 5-2). The limitation of this model is that all fractures are pre-specified by the operator in UDEC, and hence this model is not particularly generic in nature. However, this approach is useful in examining and demonstrating the broad range of support functions that shotcrete may be required to perform, and the range of shotcrete/rock mass interaction cases. In this model, shotcrete is represented using beam-elements that can resist bending and can be assigned failure properties. Tendons can also be added.

- A “generic” tunnel where a set of fractures is included ubiquitously throughout the rock mass, on a relatively fine pattern, to generically account for the development of stress induced fracturing. As the model is loaded failure occurs along selected fractures resulting in the generation of loose blocks or slabs of appropriate geometry. Two options have been considered, one in which fractures create a network of triangular blocks (Figure 5-3, top), and secondly a Voronoi Tessellation scheme in which fracture orientations are more random (Figure 5-3, bottom, and final version in Figure 5-4). In both cases appropriate regions of damage are induced, though on balance the Voronoi Tessellation scheme appears more appropriate as developed fracture patterns are more random, provided the blocks are small: in the triangular tessellation scheme the angle of the sides of the triangular blocks influences model behaviour. Again, shotcrete is included as beam elements and tendons can also be included. A description of the results of the Voronoi tessellation models form the main discussion of this report.

In all models, the following variables and conditions are catered for:

- a range in stress regimes
- changes in stress regime
- ranges in rock types
- application of dynamic loads as wave forms applied to the model boundaries
- application of internal pressures in tunnels to represent the generalised effects of support.
For all the final design modelling, it was decided to select the generic tunnel using the voronoi tessellation scheme as presented below, in which a very fine tessellation scheme is used for one diameter from the excavation boundary to permit a very fine fragmentation in the immediate tunnel walls. A coarser tessellation scheme is adopted further from the tunnel, where fractures would be expected to be less frequent in reality. A 4 x 4 m tunnel was used as a "standard" geometry.

**Figure 5-1:** Wedge model to examine shotcrete resistance to ejection of small blocks.
Figure 5-2: “Realistic” tunnel model in bedded strata.
Figure 5-3: Preliminary “Generic” tunnel models using a triangular tessellation scheme (top) and voronoi tessellation scheme (bottom). The latter was selected as a best-estimate representation for this analysis, due to the more random nature of fracture paths and greater irregularity of blocks generated.
Selection of rock mass properties for models

The Voronoi Tessellation model effectively represents a rock mass that initially is homogeneous and isotropic, and then fails by breaking those inter-block contacts that become over-stressed. Naturally occurring geological structures, such as joints and bedding, are not included in the model, and a decision was made that different rock mass conditions (strength and structure) could be represented broadly using a rock mass rating approach – i.e. a single strength criterion can be applied to account for the combined effects of rock material strength and structure. If, for example, cohesion and friction angle properties are defined for a Mohr-Coulomb constitutive behaviour, these parameters can be applied to both the internal material of the blocks (which alternatively could be made elastic), and for the partings between the blocks. Then, when the generalised rock strength is exceeded both the blocks and contacts may fail resulting in modelled rock mass fragmentation.

Figure 5-4: Final “generic” tunnel model showing boundary conditions of model.
For the purpose of this analysis, where the objective is to accumulate a range of tunnel deformations and support pressures for a range of geotechnical conditions, the rock masses that tunnels are developed through in South African gold and platinum mines have been broadly subdivided into five main categories: quartzite, lava, shale, pyroxenite and norite/anorthosite. For each of these a set of in-situ rock mass properties was derived by applying a methodology derived by Hoek (2002), and made readily available in the free software ‘Roclab’ available from Rocscience at www.Rocscience.com. This methodology makes use of base properties for laboratory Uniaxial Compressive Strength (UCS) and Rock Mass Rating (Geological Strength Index, GSI), a Hoek-Brown parameter, \( m_b \) that defines the shape of the curved failure envelope and a disturbance factor \( D \), which are then used to estimate an in-situ Hoek-Brown Strength envelope. The factor \( D \) has a value of zero in massive intact rock, and 1 where highly disturbed. Mohr-Coulomb properties, cohesion and friction angle are determined using a best-fit straight line to the curved Hoek-Brown strength enveloped over a determined range in confining stress.

In situ strength in the models is defined in terms of a Hoek-Brown strength criterion where:

\[
\sigma_1 = \sigma_3 + UCS \left( m_b \frac{\sigma_3}{UCS} + s \right)^{0.5}
\]

Here \( \sigma_1 \) and \( \sigma_3 \) are the major and minor principal stresses, and \( m_b \) and \( s \) define the shape of the failure envelop. These are related to GSI, and the initial, \( m_i \), value as follows:

\[
\frac{m_b}{m_i} = \exp \left( \frac{GSI - 100}{28 - 14D} \right) \quad \text{and} \quad s = \exp \left( \frac{GSI - 100}{9 - 3D} \right)
\]

The derived rock properties used in the models are listed in Table 5-1. The base properties represent the average values from available data. Note that the Hoek-Brown \( m_i \) values are estimates based on known values for similar rocks.
In Table 5-1 the base (laboratory) properties for each rock mass category are listed. In Table 5-2 the cohesion and friction values are based on calculations over a confining stress range of 0 MPa to 20 MPa. Properties are calculated for a range in Geological Strength Index (GSI) values of 20, 30, 40, 50, 60, 70, 80 and 90 and modelled. The cohesion and tension values decrease exponentially with a decrease in GSI value, whereas the friction angle has an almost linear relationship with respect to the GSI of the rock mass. These relationships with GSI are shown in Figure 5-5 to Figure 5-7.

**Table 5-1: Matrix of base input parameters**

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<thead>
<tr>
<th></th>
<th>Siliceous Quartzite</th>
<th>Strong Lava</th>
<th>Shale</th>
<th>Pyroxenite</th>
<th>Norite/Anorthosite</th>
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<tr>
<td>UCS</td>
<td>200</td>
<td>250</td>
<td>150</td>
<td>100</td>
<td>150</td>
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<td>2900</td>
<td>2700</td>
<td>3200</td>
<td>2800</td>
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</tbody>
</table>

**Table 5-2: Appropriate cohesion, friction and tension values for rock mass with GSI values ranging from 20 to 100.**

<table>
<thead>
<tr>
<th></th>
<th>Siliceous Quartzite</th>
<th>Strong Lava</th>
<th>Shale</th>
<th>Pyroxenite</th>
<th>Norite/Anorthosite</th>
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<td>Cohesion</td>
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<td>Friction Angle</td>
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<td>52</td>
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<td>47</td>
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Figure 5-5: Rock mass tensile strength as a function of GSI value.

Figure 5-6: Rock mass friction angle as a function of GSI value.
Figure 5-7: Rock mass cohesion as a function of GSI value.
Initial models were run in which elastic properties (Bulk and Shear Moduli) were down rated by a consistent 80% from the values listed in Table 5-1. After consideration it was felt that although this is done for general overall parameters used in mine-wide modelling (e.g. for Minsim or MAP3D-type analyses), this was inappropriate for these UDEC models and the modulus should be representative of the GSI. Elastic properties for the in-situ rock mass were then estimated using the following empirical method as proposed by Hoek and Diederichs, 2005. The modulus of the rock mass is a function of the GSI value of the rock mass as well as the laboratory tested Young’s Modulus of the rock. Elastic properties are listed in Table 5-3. The equation is presented below:

\[ E_{rm} = E_i \left[0.02 + \frac{(1-D/2)}{(1 + e^{(60+15D-GSI)/11})} \right] \]

Where:
- \( E_{rm} \): Down rated rock mass modulus
- \( E_i \): Intact modulus of rock
- \( D \): Disturbance or damage factor of the rock mass (For a loosened rock mass \( D = 1 \), for an undisturbed rock mass \( D = 0 \)). For the purpose of this exercise \( D \) was chosen as 0.3 for all rock types based on the blast damage from previous underground observations.
- \( GSI \): Geological Strength Index

Using appropriate values for Poisson’s Ratio, Bulk and Shear moduli are then calculated.

A number of approaches were initially pursued to determine the most appropriate method of determining shotcrete requirements and these are presented in section 5.2. The main output is presented in the subsequent sections. The influence of de-stressing, squeezing conditions and tunnel size was investigated and a system is proposed, applying the ground reaction charts.

**In-situ stress conditions examined**

Models were run with a variety of in-situ stress conditions. Different in-situ stress conditions were examined in the typical “gold” or “platinum” rock masses.

For a the “gold” rock masses (quartzite, shale and lava) field stress conditions of 60 MPa, 80 MPa, 100 MPa and 120 MPa.
Field stress conditions of 16 MPa, 32 MPa, 48 MPa and 64 MPa were applied to the norite/anorthosite and pyroxenite rock masses (based on what is currently experienced within the Bushveld complex).
### Table 5-3: Down rated Moduli for different rock types

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5.2 Preliminary numerical model results – comparison of model “laboratories”

5.2.1 Initial model results: simple wedge model

Results of the simple wedge model are discussed here briefly. The objective was to examine the shotcrete properties required to prevent the ejection of a wedge of ground from the wall of a tunnel.

The intention was to approximate a face-burst – underground experience indicates that a thin layer of poorly applied shotcrete to the face of a newly excavated tunnel in a high stress environment is adequate to reduce the incidence of face-bursting. Very often these face bursts leave a circular or oval damage area in the face, with both hackly and smooth zones (mirror zones) due to a slab popping from the wall as a result of the sudden and very rapid development of a fracture. It was felt that this geometry could be approximated as a thin wedge on the side of a block in two dimensions in UDEC, where the angle of the wedge surfaces relative to the block face is half the characteristic friction angle for the rock material being examined.

Example results for one of these models are shown in Figure 5-8. The upper plot shows an unsupported wedge being ejected, while the lower plot shows a thin (2cm) layer of shotcrete restraining it. In this instance the wedge is 1 m high and 0.24 m thick at the widest point.

Some variation in size and geometry of the typical wedge shown in Figure 5-8 was experimented with, the intention being to look at burst areas up to approximately 2 m in height in various rock types.

The rock mass was assigned elastic properties, with typical intact rock (laboratory) properties assigned to the partings forming the wedge. The modelling sequence was to create the glued wedge geometry within a confined block under a stress field where the vertical component is 250 MPa (typical of the stress concentration in the face, or wall, of a deep tunnel). The second step was to remove the confinement on the left of the model, representing the excavation of the tunnel. Depending on rock properties, the 250 MPa vertical stress should be adequate to cause the wedge surfaces to fail. Upon failure cohesion and tensile strength is reduced to zero (to reduce strength in the event that there is ongoing contact between faces across the parting) and the wedge is ejected. Models were run initially without a shotcrete layer (to ensure that ejection occurs) and were then re-run with various thicknesses of layers. For the layer to still be ejected
if the shotcrete is inadequately strong, constructional partings with strength the same as the shotcrete material were inserted through the shotcrete layer at the edges of the wedge. A brief examination of stresses generated suggested these should be at 45 degrees, however it was recognised that this could strongly influence results and would have been investigated further if this set of models was more extensively pursued.

Typically the models run represented a quartzite rock mass, with UCS of 220 MPa, Cohesion of 40 MPa, Friction Angle of 52 degrees, Young’s Modulus of 75 GPa, Poisson’s Ratio of 0.2 and density of 2700 kg/m³. Wedge angles to the vertical were inclined at half the friction angle, giving, for example, a maximum width of 0.24 m for a 1 m high wedge. The model, being two dimensional, effectively represents a metre thick slice, out of the plane of the model.

Under a vertical stress of 250 MPa, the model showed that the total strain energy stored in a 1 m high quartzite wedge immediately prior to failure is approximately 38 kJ (per metre out of the 2D model plane). Initial ejection velocity is typically up to 15 m/s (8.6 m/s is shown in Figure 5-8 when unsupported. Note that these velocities are influenced by damping parameters used in the models.

When shotcrete layers were added, it appeared that to restrain this size of slab with this magnitude of velocity and energy required a shear strength approaching 200 MPa in a 25 mm layer, and 50 MPa in a 100 mm layer. While not orders of magnitude wrong, these results appear to be significantly higher than the strength of most actual shotcrete materials. In addition if the shotcrete layer was made elastic, ejection of the wedge induced an average stress of approximately 100 MPa on the interface between the shotcrete and rock surface over a distance of 0.1 m from the edge of the wedge. This also seems somewhat high for a bond strength.

The results shown in Figure 5-8 appear simple and neat, however the analysis was not pursued with large numbers of models because the analysis seemed flawed: first the real situation is three dimensional and the model under-estimates the true forces and energy changes involved; second the models suggested that very high strength thin layers would be required to prevent bursting, which is inconsistent with underground observations, and hence it was felt that the model was not an adequate representation of the mechanics of the true problem; lastly model results can be strongly influenced by choice of properties and damping parameters.
Figure 5-8:  Comparison of ejection of unsupported versus supported wedge of ground in a simple wedge model – quartzite loaded to 80 MPa.
5.2.2 Initial model results: “realistic” tunnel in bedded strata model

The objective with the “realistic” tunnel model was to create a model tunnel where the rock mass captures the range of natural geological and stress induced partings that are observed around tunnels in deep mines. These would include bedding, joints, and stress fractures: all those features that create blocks that may require support around an excavation. One of the limitations with UDEC is that all partings have to be pre-defined by the modeller – blocks cannot be split during a model run other than on a parting that is pre-defined. So, to create a stress fracture pattern around a tunnel the model must first estimate what fractures the pattern may comprise, and insert these in the model during model construction.

A first attempt at this type of geometry was illustrated in Figure 5-2, above, where the fracture pattern and velocities on blocks are indicated, and suggest that a large joint and bedding-bound wedge in the hangingwall is collapsing. Figure 5-9 shows the stress distribution in the same model, indicating that there has been yield in the sidewalls and high stresses are re-distributed to an area outside the defined stress-fracture pattern.

Figure 5-9: “Realistic” tunnel model in bedded strata showing the induced stress regime and “softened” region
The effect on stress distribution in Figure 5-9 appears very realistic, however it is apparent that the stress distribution is strongly influenced by the defined fracture pattern – with distinct differences in stress magnitude on either side of the tunnel. The result is that it would be very difficult to turn this into a design tool to examine the loading induced in shotcrete under a wide range in conditions. The result is that a design model is required that can be less biased by modeller-defined (or chosen) fracture patterns – hence the development of generic models with extensive fracture tessellation patterns throughout.

5.2.3 Initial model results: “generic” tunnel using in bedded strata model

As noted above, both triangular and voronoi tessellation patterns were experimented with in the development of a model containing sufficient possible fracture paths that the development of fragments in the rock mass is more a function of model response to over-stressing the rock mass, and less the effect of user-defined patterns. The voronoi pattern was selected as preferable.

As an example of the first voronoi models, which were half-symmetry, Figure 5-10 shows the fragmentation and stress distribution pattern around a 3 m tunnel in a 100 MPa stress field. Considerable damage, resulting in fragments of varying size and shape, occurs in the tunnel sidewall. There are open fractures extending into the rock mass for several metres from the tunnel skin, and stresses are clearly redistributed as a result of the rock mass failure. The model appears to provide a reasonable representation of anticipated overall rock mass behaviour. These results lead to the choice of a generic voronoi tessellation model as the analytic tool of choice.
Figure 5-10: Half symmetry “Generic” tunnel model in a rock mass containing a fine voronoi tessellation pattern as a pre-defined potential fracture network, showing the induced stress regime and “softened” region

5.3 Final voronoi tessellation model results – model calibration – comparison to an underground case study

5.3.1 Calibration objectives

One of the problems with running large numbers of numerical models as a basis for developing guidelines in real mining situations for different geotechnical conditions, is that models only reflect the input properties used, which may or may not be a good reflection of real-life circumstances. It is therefore important to compare model results to some actual underground observations in order to provide some measure of calibration of model results, if they are to be in any way trustworthy.

One of the limitations of the sites selected for underground monitoring as part of this project (Chapter 4) was that either the stress changes were small, or mining geometry was complex.
Consequently, as part of this numerical modelling exercise, an initial model calibration was made against measurements from an earlier SIMRAC project at an instrumented site in a tunnel at a depth of approximately 2300 m below surface in Elsburg quartzites at Kloof gold mine (Haile et al., 1998).

5.3.2 Calibration case study site description

This site comprised a simple tunnel which was subjected to a gradual stress increase due to stoping activities in the vicinity, and periodic dynamic loading from distant seismic events. Support in this area consisted of 2.2 m, 16 mm rebar rock bolts on a 1.5 m diamond pattern, which applied a support pressure of approximately 60 kPa. The tunnel instrumentation consisted of extensometers located at the mid-point of each sidewall and the centreline of the hangingwall. This configuration measured what was considered to be the maximum anticipated deformation and extent of damage to the rock mass. The total measured closure of the sidewalls is illustrated in Figure 5-11, which indicates that, as stoping progresses, the rate of deformation of the sidewall is approximately 5 mm per 1 MPa change in vertical stress. This rate is comparable to the deformation rate from a Hartebeestfontein case study and is approximately double the rate determined from another site at Buffelsfontein mine (Haile et al., 1998). The ‘jumps’ in deformation in Figure 5-11 were associated with seismic events, the first of which was a magnitude M=1 at a distance of approximately 130 m from the site and the second of which was a magnitude M=2.5 at a distance of approximately 200 m. These seismic events would result in theoretical ground velocities (no amplification) of approximately 0.03 m/s and 0.13 m/s respectively. The second seismic event was associated with physical damage of a shakedown nature at the experimental site. The result of the seismic events was that the deformation of the sidewall closest to the source of the seismic event is approximately three times that of the sidewall in the ‘shadow’.
Figure 5-11: Measurements of total closure of the sidewalls at the Kloof site (after Haile et al. 1998).

Figure 5-12: Extensometer results for ‘up-dip’ sidewall at Kloof site (after Haile et al. 1998).
Figure 5-12 illustrates the dilation profile of the rock mass in the ‘up-dip’ sidewall of the tunnel. Of significance at this site is the ‘opening’ within the rock mass between 1.5 m and 2 m, which could be an indication of the potential depth of unstable ground requiring containment by support. The site showed that development of deformation around the tunnel is orientated perpendicular to the applied major principal stress. The extent of the zone of deformation was also influenced by the bedding structure, being greater parallel to the bedding. Very limited deformation occurred in the hangingwall and footwall of the tunnel.

5.3.3 UDEC calibratory model comparative results

A model using a voronoi tessellation scheme, an internal support pressure of 50 kPa, roughly equivalent to the case study site, and representing a quartzitic rock mass with GSI of 70 was run under a range of stress conditions taken as similar to the case study described above. Note that while the property derivation methodology outlined above is correctly applicable only to continuum models, where the discrete effect of discontinuities is omitted, it was considered to be a reasonable method to apply here as the intention was not to accurately represent the initial rock mass complexity (including joints and bedding) but to have a rock mass whose properties would be broadly representative of overall rock mass properties and would, unlike a continuum model, break into discrete fragments after failure, resulting in very local loading and deformation to any applied coating. The aim in this calibration exercise was to check that the selection of GSI, properties and field stress, produced deformations in the same “ball-park” as measured underground values.

Typical model behaviour is illustrated in Figure 5-13 and Figure 5-14, showing stress redistribution and deformation examples from where moderate failure occurred – in a quartzite with rock mass rating (GSI) of 70, under a stress field where the vertical component is 80 MPa and horizontal is 40 MPa. The models clearly show fracture development, with sidewall-parallel stress fracture development resulting in a slabbed sidewall. Stresses are redistributed from damaged areas to confined and solid rock deeper into the sidewall.

The Kloof case study above indicated a total sidewall closure of between 40 and 80 mm across the tunnel, with an average of 55 to 60 mm at the points recorded. Figure 5-15 shows the model recorded 20 mm in the right wall and up to 60 mm on the left wall, with the difference resulting from local voronoi tessellation geometry and fracture paths developed. Depending on
where total closure is measured in the model it is approximately 80 mm at maximum, and 60 mm at mid tunnel height. This is remarkably similar to the Kloof case study, and gives confidence to the method of property selection, and voronoi tessellation model scheme.

*Figure 5-13:* Example UDEC model showing fragmentation of the sidewall and stress redistribution in response to rock mass failure – model represents quartzite with GSI of 70 after a field stress increase from 80 to 100 MPa
Figure 5-14  Example UDEC model showing horizontal movement contours in response
to rock mass failure – model represents quartzite with GSI of 70 after a field
stress increase from 80 to 100 MPa

Based on this calibration, and comparison to the measurements at the sites selected as part of
this project, it was decided that the models and selected properties provide a reasonable
approximation of underground measurements. This provides confidence in the model results for
a wide range in conditions, and permits the development of models under a range of conditions
which could be adopted as being generally representative.
5.4 Representation of the effect of support in models and an outline of the GRC methodology

5.4.1 Explicit representation of support in UDEC

UDEC can explicitly represent both tendons and shotcrete in models, using one-dimensional cable elements, with properties representing the bar and rock bond, and beam elements with an elastic yield element for the shotcrete. An interface can be placed between the beam and the rock surface, to represent the effect of limited rock-bond strength.

Figure 5-15 and Figure 5-16 show example plots from models in which bolts and shotcrete were explicitly included. Bolts are loaded where the rock mass fractures and yields. Similarly the shotcrete, while loaded down the entire sidewalls shows local peaks in load where the slabs are largest.

Figure 5-16 shows the horizontal deformation applied to the shotcrete in each wall. The deflection is relatively smooth and there appears to be no bulking between bolts. Deformation in the left wall is greatest towards the base due to a large slab.

One of the limitations of running many UDEC models with support included is that the permutations, such as thickness, strength and deformability of shotcrete, are endless, particularly when it is necessary to consider behaviour in a wide range in rock types. Changes to any of these shotcrete parameters can result in notable changes to rock mass deformations and it is not a simple matter to examine the support pressures generated by a shotcrete layer in the model, as during the time-marching computational process the reactive forces applied by a particular shotcrete material develop in response to rock movement and in turn limit movement. It is difficult to isolate shotcrete system capacities. Further the final loads and deformations are influenced by the amount of deformation permitted prior to shotcrete application in the model.

Consequently a simpler approach was sought and it was decided to adopt a Ground Reaction Curve methodology to identify required support pressures and tunnel wall deformations as a function of rock type, GSI and applied stress field and excavation size. Knowing deformation and support pressure it is then feasible to identify shotcrete requirements separately using yield line theory.
Figure 5-15: Example UDEC model showing stress induced in 100 kN capacity bolts and a 25mm shotcrete layer after a field stress increase from 80 MPa to 120 MPa
5.4.2 Ground reaction curve methodology

A Ground Reaction Curve (GRC) is a graph that relates closure across a tunnel to the confinement, or support pressure applied to the internal tunnel walls. It was originally designed to represent the relaxation of tunnel walls close to the face of a tunnel during excavation (Hoek and Brown, 1984). Graphs for the development of support pressure by support units in response to deformation can be overlain on the GRC, and estimates made of the ability of support to limit deformation, or the likelihood of support becoming overloaded. Support pressure load-lines can be developed for various types of tendon and linings such as shotcrete.

To develop Ground Reaction Curves the methodology involves replacing the rock mass in the tunnel with a set of forces that initially equal the applied field stress and are then reduced incrementally to zero. Each increment represents a level of support pressure, and at each increment the closure across the excavation (or the deformation of one wall) can be monitored.
as can the extent of rock mass damage. The GRC is then a graph of the internal pressure applied, plotted against excavation closure. A schematic example of a GRC is shown in Figure 5-17.

Key aspects of a GRC are indicated in Figure 5-17. At high support pressure the rock response is linearly elastic, with a linear increase in deformation as support load is relaxed. At some critical support confinement pressure rock failure is initiated and the response is no longer linear, with the rate of deformation starting to increase. At some point (particularly in a high field stress) the deformation increases exponentially, indicative of a critical level of support pressure below which the rock mass will unravel.

Figure 5-17: Schematic diagram showing a typical Ground Reaction Curve (GRC)

5.4.3 Accounting for stress change in GRC models

In needs to be noted that the GRC models all represent a single tunnel, run under a constant stress state. In other words, the model is set up with a constant field stress condition, the tunnel is excavated and the internal support pressures are relaxed to zero. To create a ground reaction curve the field stress cannot be altered during the model run.

In a mining environment, many tunnels undergo a change in stress during their period of use. This may include stress increase followed by de-stressing as a result of mining abutments or over-stopping. It is assumed that the effect of stress change can be accounted for by moving from, for instance, a GRC developed under a field stress of 80 MPa, to one developed under a field stress of 120 MPa.
To check whether it was possible to examine the effect of stress change by moving from one static condition GRC to the next, or whether there was considerable stress path dependency, models were run where a tunnel in quartzite was initially excavated in an 80 MPa stress field, then stress was increased progressively to 100 MPa, 120 MPa, then reduced to 60 MPa. Sidewall deformations are plotted against those derived from the equivalent GRC curves in Figure 5-18.

When the field stress is increasing there is strong similarity, however deformations are not recovered when stress is reduced. As a result of damage under the higher stress condition, a stress reduction (with a support pressure of 0.1 MPa) results in rock mass unravelling, and an increase in deformation.

Further assessments of the effect of stress reduction and the influence on support requirements are provided in following sections. The broad conclusion is that in an increasing field stress situation it is feasible to move from one GRC to the next. However in a decreasing field stress environment it is not, because the higher field stress has already done damage to the rock mass. It is difficult to account for this by moving to a GRC for a lower GSI, as the rock damage (and hence GSI reduction) applies only to a limited volume of the rock mass around the excavation.
Figure 5-18: Comparison of deformations resulting from changing stress (a stress path) applied to a single model versus those from GRC curves derived individually under static stress conditions

5.4.4 Equivalence of GRC and explicit support pressures

In designing support, and in using a GRC, we generally assume that the support pressure generated by a support system is typically equivalent to the load capacities of the support units, divided by their spacings. For example, if 10 ton (100 kN) capacity rock bolts are installed on a 1 m spaced pattern, the capacity of the support system is 100 kN/m² (100 kPa, or 0.1 MPa). If shotcrete is included its support resistance is a function of its resistance to bending or flexure.

One of the questions that arose during the project was whether this is in fact correct, and does the pressure applied in a GRC model provided a fair representation of support pressures achievable by installed support. In the GRC model the effect of support is represented as an equally distributed pressure across the walls of the tunnel. In practice tendons provide point loads, and shotcrete may achieve a fabric providing containment between bolts – potentially very dissimilar to a uniformly distributed load.
A model was set up with 100kN support capacity tendons installed on a 1 m spacing around the 3.5 x 3.5 m tunnel interior. This gives a theoretical support capacity of 100kN/m², or 0.1 MPa. Rather than fail after reaching yield load, the modelled bolts were allowed to continue to apply a 100kN load.

The model was run with a quartzitic rock mass, GSI of 70, starting with a vertical field stress of 80 MPa, and put through a cycle of stress increase first to 100 MPa, then 120 MPa, then a vertical field stress decrease to 60 MPa. The sidewall deformations in the supported model are compared in Figure 5-19 to those for similar models with either no support (worst case) or an evenly applied 0.1 MPa support pressure. The support restricts deformations significantly relative to the unsupported case, but permits up to 20% greater deformations than the evenly supported case. The addition of an elastic shotcrete layer to the bolting pattern results in an even closer match with the evenly supported case.

![Effect of stress change](image)

*Figure 5-19: Comparison between modelled deformations where support is explicitly represented to those with an even 0.1 MPa support pressure*
5.5 Development of Ground Reaction Curves as a measure of support requirements – effect of GSI, stress change and tunnel size

5.5.1 Comparative effect of stress magnitude and GSI on deformation

Overall a large number of voronoi tessellation models were run, for the full range in rock types and GSI values using the properties derived in section 5.1.4, above, under a range of stress conditions. In each case, a Ground Reaction Curve (GRC) was developed by progressively relaxing pressures applied to the interior of the excavation. A library of GRC curves is provided in a separate appendix to this report. Rather than relating each GRC to a field stress, it was decided to relate GRCs to the maximum stress concentration in the excavation wall. This eliminated the need to run a very large number of field stress cases other than ones where the k ratio was 0.5. Very approximately the maximum stress concentration was estimated using the formulae for concentrations around a circular opening, using the major ($\sigma_1$) and minor ($\sigma_3$) field stress components in the plane of the cross section:

\[
\text{Peak stress concentration} = 3 \sigma_1 - \sigma_3.
\]

The complete set of results of the analyses, for each rock type, is presented in Appendix C in volume II of the report.

As examples of the overall process, the results are presented in this section for models representing a quartzitic rock mass with GSI ranging from 20 to 90, under field stress conditions of 60, 80, 100 and 120 MPa.

A comparison of the rock wall damage occurring around a 3.5 m tunnel for quartzitic rock mass GSI values of 20, 30, 40, 50, 60, 70, 80 and 90 under a low stress field (60 MPa) is shown in Figure 5-20. For GSI values between 60 and 90, very little deformation is experienced. For GSI values below 60, the deformation increases significantly. The deformation at the centre of the sidewall and hangingwall is used to plot the ground reaction graphs for different GSI values, rather than closure across the excavation.

The Ground Reaction Curves (GRC) for these 3.5 m tunnels sited in a quartzite rock mass are presented in Figure 5-21 to Figure 5-28 for GSI values of 20, 30, 40, 50, 60, 70, 80 and 90, for each stress field – 60, 80, 100 and 120 MPa (Maximum stress concentrations of 150, 200, 250 and 300 MPa).
All GRCs show a similar structure. Initially, while the rock mass is elastic and undamaged, the relationship is linear at high support pressure. However below a critical level of support pressure confinement some stress-induced failure of the rock mass is initiated and the rate of deformation increases, resulting in a curved relationship. In general this failure initiation point and deviation from elastic behaviour requires a support pressure of (for example) 10 MPa or more. This is a not practical to achieve with normal support technology, and hence support is not intended to prevent stress-induced rock failure.

In each graph, as support pressure is reduced further, approaching zero, the rate of deformation progressively increases. At some low level of support pressure confinement this deformation rate appears to increase exponentially/rapidly where support pressure is unable to prevent the fractured rock mass unraveling and collapsing. This is generally an achievable level of support pressure.

In addition to identifying a level of support pressure required to prevent final collapse and unraveling, the GRC graphs provide an indication of the level of deformation that may be applied to support. If an estimate is made of the amount of deformation that may have occurred prior to support installation, the graphs provide the remaining deformation that may be applied to the support system as it develops a resistive load. An estimate can be made of the amount of deformability required of support, and whether particular support systems may be liable to fail in a given environment. Note that the GRC support pressure-deformation relationship varies as a function both of rock mass strength and applied stress field. Graphs shown in Figure 5-21 to Figure 5-28 compare quartzite conditions and responses.
Figure 5-20: Visual comparison of the rock wall deformation around tunnels for different GSI values at 60 MPa field stress. For GSI values between 60 and 90, very little deformation is experienced. For GSI values below 60, the deformation increases significantly.
**Figure 5-21**: Quartzite (GSI 20) Ground Reaction Curves (note that the models exhibited squeezing behaviour at the higher field stresses)

**Figure 5-22**: Quartzite (GSI 30) Ground Reaction Curves (note that the models exhibited squeezing behaviour at the higher field stresses)
**Figure 5-23:** Quartzite (GSI 40) Ground Reaction Curves

**Figure 5-24:** Quartzite (GSI 50) Ground Reaction Curves
Figure 5-25: Quartzite (GSI 60) Ground Reaction Curves

Figure 5-26: Quartzite (GSI 70) Ground Reaction Curves
Figure 5-27: Quartzite (GSI 80) Ground Reaction Curves

Figure 5-28: Quartzite (GSI 90) Ground Reaction Curves
5.5.2 Effect of excavation size: comparison of 3.5 m, 5 m, and 7 m wide tunnels

Most of the models run as part of this project represented a tunnel of 3.5 x 3.5 m square cross section. Deformations obviously vary as a function of excavation size and consequently a series of models was run to examine the effect of tunnel size on GRC results and tunnel deformations and strains. For each model case, deformations have been compared at various support pressure levels.

The main features of these models include:

- Excavation sizes compared were 3.5m, 5 m and 7 m.
- Models represented excavations in a quartzite rock mass with a GSI value of 70. The minimum voronoi block size for the 7 m wide excavation was increased to 40 cm, whereas the minimum block size for the other two models was 20 cm.
- The models assumed no influence of dipping bedding planes.
- These models all make use of a static stress field comprising components of 120 MPa vertical stress and k-ratio of 0.5 i.e. no change in stress orientation or magnitude after initial development.

Results are described below. Deformations are listed in Table 5-4 for various support pressures and normalised results are shown graphically in Figure 5-29.

Preliminary modelling had shown that model deformation and failure is obviously influenced by voronoi tesselation size, particularly on the perimeter of the excavation. Fewer than six blocks per wall resulted in no choice in failure path, and all partings failed. With eight or more blocks there was a greater tendency to develop slabs, leaving some contacts between blocks unfailed. With this in mind it was considered appropriate to use a 20 cm block size as far as possible in most models, but in larger excavations it was felt acceptable to increase block size, provided it did not interfere with slabbing behaviour and left a choice of fracture paths to create. The influence on deformation was not thoroughly investigated but, within this size proviso, was felt to be small.

At very high support pressures (10 MPa), there is minimal difference between the sidewall deformation of the 3.5 m, 5 m and 7 m wide excavations. At 1 MPa support pressure, the sidewall deformation starts to be clearly influenced by excavation size, however at 100 kPa
support pressure (which is more realistic for tunnel support currently used in South African mining industry), the sidewall deformation of the 3.5 m and 5 m wide excavations is almost identical at approximately 40 mm. The sidewall deformation of the 7 m wide excavation is approximately 60 mm, which is 1.5 times that of the 5 m wide excavation. This indicates that the amount of sidewall deformation increases rapidly for excavation widths greater than 5 m, which suggests that a higher pressure support is required to limit the extent of the movement.

The influence of excavation size on the sidewall deformation is presented below in Figure 5-29 as a function of the support pressure that is applied to the sidewalls of the excavation. The upper limit for the sidewall pressure was taken as 1 MPa whereas the lower limit was taken as 10 kPa. The Figure plots a factor for the increase in sidewall deformation compared to a tunnel of 3.5 m diameter against excavation size. The factor was obtained for the different internal support pressures of 10 kPa, 100 kPa and 1 MPa, and can be used to estimate the likely deformations for any size of excavation.

**Table 5-4: Sidewall deformation as a function of excavation size**

<table>
<thead>
<tr>
<th>Excavation Size</th>
<th>Sidewall deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Support Pressure 1MPa</td>
</tr>
<tr>
<td>3.5 m wide excavation</td>
<td>33 mm</td>
</tr>
<tr>
<td>5 m wide excavation</td>
<td>35 mm</td>
</tr>
<tr>
<td>7 m wide excavation</td>
<td>40 mm</td>
</tr>
</tbody>
</table>
Figure 5-29: The effect of excavation diameter increase on the magnitude of sidewall deformation relative to that of a tunnel of 3.5 m diameter. Lines are indicated for various levels of internal support pressure.
5.5.3 Effect of support pressure on rock failure around an excavation

Under high stress rock failure occurs around an excavation and the width of this envelope of potentially unstable ground (depth of instability) around each excavation for different levels of support pressure was determined from the models of varying excavation size, described in the previous section. The depth of instability in the sidewalls as a percentage of excavation width, is listed in Table 5-5.

Table 5-5:  Depth of sidewall instability as a function of excavation size

<table>
<thead>
<tr>
<th>Excavation Size</th>
<th>Support Pressure 1 kPa</th>
<th>Support Pressure 10 kPa</th>
<th>Support Pressure 100 kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5 m wide excavation</td>
<td>37%</td>
<td>37%</td>
<td>37%</td>
</tr>
<tr>
<td>5 m wide excavation</td>
<td>32%</td>
<td>32%</td>
<td>32%</td>
</tr>
<tr>
<td>7 m wide excavation</td>
<td>22%</td>
<td>22%</td>
<td>22%</td>
</tr>
</tbody>
</table>

For the purpose of this assessment, the depth of instability is defined as the point in the sidewall where there is a clear separation occurring in the UDEC model (i.e. joints or contacts with zero normal force or stress) or a large difference in the rate of deformation between two points in the sidewall. From the results in Table 5, we see that there is no identifiable influence of the support pressure on the depth of instability. However, there is a decrease in depth of instability in the tunnel sidewall as a function of the percent of width of excavation as the width of excavation increases. From an excavation width of 3.5 m to one of 5 m this is small. At an excavation width of 7 m, the depth of instability was however only 1.54 m (22% of excavation width), which is significantly lower, as a percentage, than that of the 5 m wide excavation, where actual depth of failure is similar, at 1.6 m, giving 32% of excavation width.

Given that the 7 m wide excavation is subjected to a larger sidewall deformation than the 5 m wide excavation, as shown in Table 5-4, the lower (22%) depth of instability for 7 m wide excavation could be a result of the difference in minimum discrete block sizes (20 cm versus 40 cm) between the two models.
When comparing depth of instability in metres i.e. 1.3 m, 1.6 m and 1.5 m for the 3.5 m, 5 m and 7 m wide excavations, the difference between 5 m and 7 m is minimal. It’s easier to argue that the small variation is due to block size.

If the extent of the unstable envelope in the hangingwall is examined, a difference in predictable behaviour is observed. Results are listed in Table 5-6. From these results, we see that there is no influence of either the support pressure or width of excavation on the depth of hangingwall instability as a per cent of excavation size.

Table 5-6:  **Depth of hanging wall instability as a function of excavation size**

<table>
<thead>
<tr>
<th>Excavation Size</th>
<th>Depth of hangingwall instability (% of width of excavation)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Support Pressure 1 kPa</td>
</tr>
<tr>
<td>3.5 m wide excavation</td>
<td>40%</td>
</tr>
<tr>
<td>5 m wide excavation</td>
<td>40%</td>
</tr>
<tr>
<td>7 m wide excavation</td>
<td>40%</td>
</tr>
</tbody>
</table>

5.5.4 **Effect of a reduction in stress field on rock mass deformation**

Most of the models run were as described in the previous section – models of tunnels, run under a static stress field, where a ground reaction curve (GRC) is derived by progressively reducing support pressure. However, there are many cases underground where support is installed under other conditions, such as where the stress field decreases, or changes in orientation after excavating the tunnel, or severe squeezing conditions (excessively high stress/strength ratio) are encountered. The analyses described in the sections below cover these more complex conditions.

The influence of de-stressing a tunnel was investigated in UDEC by reducing the field stress applied as model boundary conditions from an initial higher state and monitoring the influence on the resultant rock wall deformations. Models of this type were run with constant internal support pressures of 1 MPa, which is thought to be considerably higher than current support in the South African Mining Industry, and 1 kPa which is somewhat less than what current typical tendon support is considered able to provide. In other words these values bracket the possible
support system cases for support installed prior to stress changes occurring around the excavation. The following were investigated:

- The effect of de-stressing a tunnel on rock wall deformation on the skin of the excavation versus a point inside the rock wall
- The effect of de-stressing a tunnel under high and normal support pressure on depth of instability

The results were compared to underground case studies.

**Extent of the envelope of unstable ground in excavation walls**

When field stress is reduced around an excavation it is anticipated that rock mass confinement is reduced, and more reliance is placed on the effectiveness of the support system to keep potentially loose blocks in place. An attempt has been made to identify this in the models by examining changes to the region of damaged ground around the modelled excavations. There are a number of different parameters that can be used to indicate the width (or depth) of the envelope of ground around an excavation that becomes unstable as a result of stress-induced damage, and hence requires support. Note that the volume that is actually unstable is not necessarily the same volume as the damage zone.

In these voronoi tessellation UDEC models where the rock mass can fragment, deformation is one indicator. Examples, showing the modelled extent of the deformation in the horizontal direction (for cases where low (1 kPa) and high (1 MPa) internal support pressure is applied to the rock walls), which gives an indication of the depth of instability into the sidewall, are presented below in Figure 5-30 and Figure 5-31. Without support, open fractures result in sharp differences in deformation, moderate on one side, high on the other. With higher internal support pressure applied the overall sidewall deformation is reduced at the same point in the sidewall by up to 4 cm, which is significant especially if shotcrete support is installed, and sharp displacement changes are eliminated.
**Figure 5-30:** Horizontal deformation around tunnel under low support pressure of 1 kPa.

**Figure 5-31:** Horizontal deformation around tunnel under high support pressure of 1 MPa.
Another indicator of the extent of the unstable envelope in a UDEC model is provided by volumetric strain contours. Plots of the volumetric strain for 1 MPa and 1 kPa applied stress conditions are presented below in Figure 5-32 and Figure 5-33. These plots indicate that at lower applied support pressures inside the tunnel then volumetric strain that results from destressing is greatly increased from 1.5% strain under a support pressure of 1 MPa to 4% strain if support pressure is only 1 kPa.

If the point of zero change in volumetric strain is taken as the limit of potential instability, it can be seen that the extent of instability may increase from approximately 1 m into the sidewall to approximately 2 m, in response to de-stressing. This illustrates the importance of effective support, and provides some indication of the risk of collapse as a result of stress change, if support is inadequate.

**Figure 5-32:** Volumetric strain around tunnel as a result of de-stressing of the tunnel under high support pressure of 1 MPa.
DE-STRESSING UNDER LOW SUPPORT PRESSURE (< 1 kPa)

Figure 5-33: Volumetric strain around tunnel as a result of de-stressing of the tunnel under low support pressure of less than 1 kPa.

Influence on deformation of sidewall skin versus a point inside the rock wall

The models show that changes in deformation as a result of changes in stress are very path dependent. The differences in deformation that may occur on the skin of an excavation relative to a point deep into the excavation sidewall are strongly influenced by the level of support pressure applied prior to field stress change.

In its simplest sense, in situations where the rock mass becomes de-stressed, and field stress is reduced, the modelling shows that there is a significant inter-relationship between the resulting change in deformation and the magnitude of support pressure that was applied prior to field stress reduction. Note that for most of the cases examined in these models, de-stressing, or field stress reduction, generally induces an increase in deformation into the excavation. A reduction in field stress will result in a reduction in elastic deformation, however in most of the cases here the rock mass is damaged and fragmented. The result is that a reduction in field stress results in a reduction in confinement to the fragmented rock mass. This permits the fragments to move – generally into the excavation.

Comparative graphs illustrating tunnel wall movements under various support pressure cases during a field stress reduction process are shown in Figure 5-34 to Figure 5-37.
If internal support pressures in the tunnel model are low, then larger deformations occurred under the original stress regime (than if support pressure was high) with relatively more unravelling and opening of partings. During de-stressing, as the field stress is reduced, it is possible to see larger deformations occurring further away from the tunnel than on the tunnel skin – the rock mass on the skin is already significantly unravelled but is held in place by a low support pressure. However, it should be noted that if the support pressure confinement is then removed completely (equivalent to support failure, or complete corrosion), the resulting final relaxation of the skin of the tunnel sidewall is still much higher (in a field stress of 64 MPa, up to 10 cm deformation occurs for pyroxenite with GSI value of 60) than further away from the sidewall (at a distance of 1.75 m inside the sidewall skin, deformation of approximately only 3.5 cm occurs for pyroxenite with GSI value of 60).

If the initial internal support pressure is higher under the original stress regime, less unravelling and opening of partings occurs under the original field stress. A reduction in field stress then results immediately in a slightly higher movement on the tunnel walls than at a distance into the wall. De-stressing at higher support pressures results in lower total ultimate deformations (in a field stress of 64 MPa, up to 7 cm occurs for higher support pressure compared to 10 cm for lower support pressure, for pyroxenite with GSI value of 60). The effect of de-stressing followed by failure of internal support (complete removal of support pressure) results in higher post-removal deformations than if the installed support provided a lower support pressure. Thus, to reduce the overall effect of de-stressing on the deformations around a tunnel, and risk of failure of one support component resulting in tunnel collapse, the model results suggest that it is wise to install a very good support system prior to de-stressing rather than installing additional support during the de-stressing process.
**DE-STRESSING UNDER LOW SUPPORT PRESSURE (< 1 kPa)**

**Figure 5-34:** Deformation at a point 1.75 m into the sidewall as a result of de-stressing of the tunnel under low support pressure of less than 1 kPa.

**Figure 5-35:** Deformation on skin of excavation as a result of de-stressing of the tunnel under low support pressure of less than 1 kPa.
Figure 5-36: Deformation at a point 1.75 m into the sidewall as a result of de-stressing of the tunnel under high support pressure of 1 MPa.

Figure 5-37: Deformation on skin of excavation as a result of de-stressing of the tunnel under high support pressure of 1 MPa.
Influence of magnitude of field stress reduction on changes in tunnel wall deformation

The results above looked at limited de-stressing of tunnels. For comparative purposes, the effect of de-stressing a tunnel sited in a quartzitic rock mass, starting under very high stress and ending under very low stress, is shown in terms of deformation into the tunnel of a point on the skin of the tunnel sidewall in Figure 5-38.

In this example, de-stressing of a 3.5 m tunnel from 120 MPa field stress to 60 MPa field stress in a quartzite rock mass resulted in an overall increase in sidewall deformation of 30%.

Figure 5-38: Influence of de-stressing the tunnel on the sidewall deformation from 120 MPa to 60 MPa in 20 MPa incremental steps (quartzite rock mass).
The sidewall movement assessment under an extreme range in stress change for a tunnel in quartzite, as presented in Figure 5-38, was extended to cover GSI values of 40, 50, 60, 70 and 90 for cases where a reduction in the vertical stress from 120 MPa to 60 MPa is modelled in 20 MPa incremental changes. For each case the results showed that for a GSI of 40 and 50, the tunnel exhibits squeezing behaviour even when de-stressing from a maximum field stress of 80 MPa. The change in sidewall deformation for tunnels sited in a rock mass of GSI 70 and 90 is presented below in Figure 5-39. The results show that tunnel sidewall deformation will increase by a factor of approximately 1.45 for both GSI 70 and GSI 90 rock masses when the field stress is reduced by 50%.

The effect of differences in internal support pressure (including changes during the de-stressing process), can be gauged by reference to an appropriate ground reaction curve. For a lower applied pressure the initial deformation prior to destressing would be greater, and hence the effect of destressing would be to induce larger deformations. For a higher applied pressure the converse would be true.

![Figure 5-39: Influence of de-stressing the modelled tunnel in terms of the proportionate change in sidewall deformation (quartzite rock mass at a constant internal support pressure of 100 kPa).](image)
To put the model results into some form of calibrated context, an example of the deformation history for an instrumented tunnel section from the Buffelsfontein gold mine (Haile et al., 1998) is shown in Figure 5-40. This figure indicates the deformation of the sidewall and hangingwall with respect to vertical field stress change. The sidewall deformation of both sidewalls is almost doubled when the vertical stress is halved as a result of overstoping (the host rock in this case was a well bedded dirty quartzite). This is very similar to the UDEC results for a tunnel modelled with the set of properties estimated for a GSI 60 shale rock mass (which are not considered too dissimilar to the field site characteristics).

Figure 5-40: History of deformation with vertical stress change from Buffelsfontein case study (Haile 1998).

A comparison of the modelled and underground extensometer measurement results is presented in Figure 5-41. This shows that the magnitude of deformation increase measured underground is higher than that produced by the best-approximated modelled results. However, it is interesting that the highest deformation for underground measurement results occurred within the first 10 MPa reduction in load, after which the slope of the curve for the up-dip sidewall follows a similar trend to that of the modelled results. The slope of the down-dip sidewall deformation increase curve is less than that of the up-dip sidewall. Note that there was no bedding included in the model, hence in the model both sidewalls are similar. Broadly
however, the comparison of the underground and modelled results appears to be similar and tends to confirm that model results are acceptable.

A comprehensive set of models with multi-step field stress reduction were constructed for different GSI values and rock types. The sidewall deformation increase factors for the different combinations are presented in Figure 5-42. The lowest GSI value at which the ground reaction charts are still applicable post-de-stressing for the different rock types is also indicated in the Figure (e.g. for quartzite, the minimum GSI value is 60), below which large deformation, or squeezing conditions, would be experienced during the de-stressing phase.

**Figure 5-41:** Comparison of underground measurements and modelling results (GSI 70 rock mass) for tunnel undergoing de-stressing.
Figure 5-42: A comparison of ‘sidewall deformation increase factor’ with different GSI rock types for tunnel undergoing de-stressing.

The effect of de-stressing on support requirements of the tunnels and support capability was shown to be significant from the model results and for this reason it is recommended that each tunnel that is planned to be de-stressed should be treated as an individual case to determine support requirements. It is recommended that the support be determined and installed to specification prior to de-stressing of the tunnel is commenced, which would greatly reduce the resultant deformation of the tunnel rock walls.

5.5.5 Very high stress/strength ratio (squeezing) conditions

Large and ongoing deformations (squeezing conditions) that can result in total tunnel closure may occur where there is an excessively high rock stress to strength ratio. These conditions can occur either under very high stress, or where the rock mass is weakened in some way, either through low GSI, or due to additional weak partings that permit squeezing.

Examples from the South African mining industry include tunnels at Hartebeesfontein Gold Mine (Bosman et al. 1998), where this behaviour was referred to as plastic time-dependent creep in
response to the rock mass being over-stressed. Tunnels were sited in a weak quartzitic rock mass, where the zone of yield around the tunnel was strongly controlled by bedding partings and the presence of weaker horizons (Figure 5-43). A modified voronoi tessellation UDEC model with additional horizontal bedding was therefore set up with two distinct regions of rock mass strength, where the tunnel is developed along the weak rock mass horizon and is overlain and underlain by a stronger rock mass. The initial modelled excavation was square with dimensions of 3.5 m x 3.5 m. No internal support pressure was applied to the rock walls of the excavation. The input parameters for this model are listed in Table 5-7. Field stress levels applied in the model started with a vertical stress of 80 MPa and were elevated to a final 108 MPa.

The model showed that deformation of up to a value of 600 mm per sidewall occurs, which results in total closure of the tunnel of up to 1.2 m. For a tunnel width of 3.5 m, this is almost as much as 35% closure. Figure 5-44 shows how the square tunnel has changed to a rectangular shaped tunnel as a result of closure and squeezing.

Significant movement along the discontinuities in the rock walls surrounding the tunnel is an indication of the formation of fractures around the tunnel. The discontinuities have opened up significantly up to a depth of almost 2.5 m into the sidewall of the modelled excavation, whilst lesser or minor opening has occurred up to a depth of 4 m into the sidewall. The dilation along these discontinuities has significantly increased the deformation in the sidewalls, hangingwall and footwall. Footwall heave is clearly visible in Figure 5-44. The rock walls, especially the sidewalls, are severely squeezed into the excavation.
Figure 5-43: Idealised graphical representation of tunnel deformation (after Bosman et al., 2000).

Table 5-7: Input parameters for model simulating squeezing conditions.

<table>
<thead>
<tr>
<th>Depth horizon</th>
<th>GSI</th>
<th>Bulk Modulus (MPa)</th>
<th>Shear Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stronger rock overlying and underlying the weaker rock horizon</td>
<td>60</td>
<td>14</td>
<td>10.5</td>
</tr>
<tr>
<td>Weaker rock horizon</td>
<td>50</td>
<td>7</td>
<td>5</td>
</tr>
</tbody>
</table>
Figure 5-44: Plot of horizontal displacement contours indicating displacement of up to 600 mm has occurred in one sidewall.

The depth of fracturing is indicated in Figure 5-45 and Figure 5-46 for the UDEC model that represents the weak rock Hartbeestefontein case. This compares well to the actual extent of underground damage reported by Bosman after 19 days. Note that the model stabilised at this point and did not continue squeezing, whereas the real tunnel continued to squeeze.

Figure 5-45: Plot of horizontal displacement contours indicating fracture zone around the modelled Hartbeestefontein squeezing case-study tunnel.
Figure 5-46: Plot of principal stresses in the model indicating stress concentration at a distance of more than 5 m from the sidewall of the excavation. Note the fracture zone around the tunnel where large openings along the discontinuities have occurred.

Figure 5-46 shows the distribution of stresses (in tensor form) around the model representing the Hartebeestefontein case study. It is clear that stress has been redistributed as a result of failure and the peak stress concentration is occurring deep into the sidewalls, which resulted in approximately 12 m (including the tunnel) width of the weak horizon being de-stressed as a result of failure and yield, similar to a 12 m wide stope panel. Given the magnitude of mass movement of the sidewalls it is thought that shotcrete support is unable to prevent this deformation and is probably unsuitable for tunnels under these conditions, except as a means of preventing complete unravelling of the sidewalls when applied in combination with mesh.

For each of the numerical models that has been run a Ground Reaction Curve (GRC) relating deformation to support pressure for each rock strength and rock stress combination has been derived. The numerical modelling results showed that squeezing conditions are obtained for different rock mass types under certain high stress field conditions. On the basis of this it is possible to identify applicable ground reaction curves for certain circumstances.
5.6 Assessment of deformation versus distance from face when using Ground Reaction Curves

One of the problems in applying Ground Reaction Curves derived from two-dimensional sections is in assessing how the deformation that occurs relates to distance from the face. If a support reaction line is going to be added on to a GRC that is determined from a 2D model, at what point should the support line start? How much deformation occurs ahead of the tunnel face, or close to the tunnel face prior to support installation?

The only way to obtain an indication of this is to assess the rate of deformation using three dimensional models. It was decided that the easiest way to do this was using the finite difference code FLAC3D. This is not directly comparable to UDEC as it is a continuum code, however using the sets of properties above, and as the objective was to obtain relative rates of displacement as a function of distance from the face, it was considered reasonable. It is a relatively quick model to obtain results from, and significantly less complex than UDEC’s 3D equivalent, 3DEC.

The FLAC3D model represents a 50m length of tunnel, with 30 m of solid ahead of the face. As with most of the UDEC models the tunnel cross section is 3.5 x 3.5 m. Models were run using a rock mass representing quartzite with a GSI of 70, under stress field conditions of 60, 80 and 100 MPa, and a k ratio of 0.5.

Model results are indicated in Figure 5-47 and Figure 5-48. In the first graph, the actual modelled deformations are plotted. In the second figure, deformations are plotted as a percentage of the final, long term deformation remote from the face.

Note that under lower stress field conditions where the amount of final damage is reduced, the deformation within 10 m of the face reaches magnitudes that are greater than the deformation remote from the face. This is a product of the tunnel being excavated in the model in a single step, rather than being advanced incrementally. In this case a locally higher stress concentration occurs in the tunnel sidewalls close to the face – higher than some distance behind the face. In the case of tunnels in a lower stress field this induces damage (and hence deformation) at a higher level than induced further from the face. This is less apparent under higher field stress conditions where the stress concentrations throughout are high enough to induce extensive damage and higher magnitude deformation.
Figure 5-47: Graph of modelled deformation in a 3.5 x 3.5 m tunnel as a function of distance from the face. Deformations are represented as horizontal closure across the tunnel in millimetres.
Figure 5-48: Graph of modelled deformation in a 3.5 x 3.5 m tunnel as a function of distance from the face. Deformations are represented as a percentage of the final deformation.

5.7 Application of the Ground Reaction Curves to determine the quasi-static loading demand in terms of expected tunnel sidewall deformations

Ground reaction curves were determined for 3.5 m x 3.5 m tunnels in typical Quartzite, Shale, Lava, Pyroxenite and Norite/Anorthosite with GSI values ranging from 20 to 90 under different maximum tangential stress levels (section 5.5.1). The full set of curves is presented in Appendix C and an example is included in Figure 5-49. It can be seen that most of the deformation takes place during the reduction of support pressure to 1 MPa, which is an order of magnitude greater than the typical support capacity in underground mines. If the support were installed too early, it would fail, but in practice the support systems do not fail. This would imply that most of the deformation has taken place prior to the installation of support. In section 5.6, the modelling indicated that most of the deformation occurs within the first 5 m from the face, which is probably during the blast and before any support is installed. It can therefore be
argued that the initial deformation will always take place before the support is installed, unless squeezing conditions occur.

In practice, support damage and significant deformation occurs as a result of stress changes. The maximum expected deformation can be determined by comparing the ground reaction curves for two different stress levels as indicated in Figure 5-49. In this example, the maximum expected deformation is 5 mm for a stress change from 150 MPa to 200 MPa. Maximum displacements have been determined for stress changes at a typical support pressure of 100kN/m². These are presented in Table 5-8.

![Figure 5-49: Estimating the maximum deformation from ground reaction curves (eg Quartzite GSI 70)](image-url)
<table>
<thead>
<tr>
<th></th>
<th>GSI</th>
<th>150-200 MPa</th>
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<th>250-300 MPa</th>
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<tr>
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<td>Squeezing</td>
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<td>Norite/Anorthosite</td>
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<td>4.60</td>
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<td>1.60</td>
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</table>
In the section 5.5.2, the influence of tunnel size was explored. The maximum expected displacement has been determined, should be adjusted for the actual tunnel size using Figure 5-29. The maximum expected displacement is then multiplied by the displacement increase factor.

In Section 5.5.4, the effect of de-stressing was explored. It was found that the depth of instability and deformation is very dependent on the applied support pressure. This highlights the importance of ensuring that support is installed to specification prior to de-stressing. Figure 5-42 indicates the deformation increase factor for tunnels (with an applied support pressure of 100 kPa) undergoing de-stressing.

5.8 Conclusions

Numerical modelling has been carried out to examine to the distribution of rock mass deformations around excavations, and the interaction with an application of shotcrete – both in terms of the effect that shotcrete has on rock mass damage and deformation, and how the deformation results in loading and damage to shotcrete. These analyses have been designed to complement the results from underground monitoring (chapter 4) – where numerical modelling permits easier assessment of a broader range of rock mass strength, excavation geometry, and stress magnitude and change conditions. In particular the modelling was required to identify support pressure limits for rock mass stabilization under varying geotechnical conditions, from which practical requirements for the share of support pressure that was required to be provided by shotcrete could be identified.

Several modelling approaches have been experimented with, in an attempt to develop a numerical ‘laboratory’ to investigate shotcrete behaviour and requirements. The most useful has proved to be a generic two-dimensional tunnel model that has been created with the discrete element code UDEC, using a voronoi tessellation scheme to permit examination of the effect of stress-induced rock fragmentation on support requirements. The benefit of this type of model (as opposed to a FLAC-type continuum model) is that fractures open as a result of rock failure and hence the locally high stress and strain values can be induced in a lining – rather than an automatic even distribution.

One of the key concerns when designing a shotcrete support system is how much deformation will be applied to the shotcrete after it is applied – this may be residual deformation occurring after installation, or additional deformation resulting from stress change. The model has proved
particularly useful in quantifying these deformations in a discontinuous rock mass, and the effect that support pressure has on these deformations – where it was found that the limits to support pressure provided by shotcrete were better estimated separately using yield line theory in chapter 7.

The model has primarily been used to examine the relationship between support pressure and rock wall deformation in terms of Ground Reaction Curves (GRC) for a range of common rock mass circumstances and stress regime conditions that occur in both the gold and platinum/chrome tabular mines.

A summary of maximum sidewall deformations is presented in Table 5-8, with GRCs in Appendix C, derived from models of 3.5 x 3.5 m tunnels. Both movements and GRC graphs are identified by rock type, rock mass rating in the form of GSI, and an estimate of the maximum elastic stress concentration that could be induced in the tunnel wall under the applied field stress regime. This stress concentration seemed a more useful identifier, and cause of damage, than any of the three principal stress components that make up the in-situ field stress.

For other excavation sizes a scaling factor (shown in Figure 5-39) can be applied – this factor was identified through running models of a selection of excavation sizes for a limited number of geotechnical conditions.

The GRC graphs are each developed for a static stress field. Consequently, the models were also used to examine the effect of field stress change on the deformations that occur when a constant support pressure is applied. It was found that if field stress is increased, then the deformation increases to a value that corresponds to the deformation anticipated by GRC derived from the final field stress. However if the field stress was reduced, if the rock mass had been damaged, then there was often a small increase in deformation resulting from reduction in confinement – suggesting more work is required of support.

Models were run with tendons and shotcrete individually represented and included. These tended to confirm that deformations were similar to those experienced when applying simple support pressures to the interior of the excavation, if the system support pressure was estimated based on the pull out load of the bolts, and resistance to flexure of the shotcrete. These models confirmed that the use of the GRC methodology to estimate support requirements was reasonable.

The model results, in particular the GRC graphs, can be directly used to estimate design support requirements. For selection of appropriate shotcrete systems for a tunnel in a certain
rock mass under an estimated stress field, the Ground Reaction Curves can be used to supply
estimates of required support pressures to limit deformations to levels that are non-destructive
to the support. The GRC graphs also provide a measure of the maximum movements in the
tunnel wall (before complete disintegration), should support prove ineffective. For specific
excavation designs, considerations and modifications to apply to the basic GRCs include
scaling factors for excavation size, potential for extreme squeezing (deformations are large and
relatively indeterminate), and whether the tunnel will be subjected to stress change, including
destressing.

Comparison of model results to the underground field sites tended to confirm that model results
indicated deformations and support pressure requirements of the correct order of magnitude.

With excavation support requirements defined in terms of support pressure and deflection (by
selecting the appropriate GRC and modifications for an excavation under consideration),
estimates of required shotcrete strength, thickness and deformability can be made using yield-
line theory, as reported in chapter 7.

To make use of the GRC graphs that have been derived from the modelling, the following input
parameters need to be specified:

- Rock Type
- Field Stress
- Internal Support Pressure limits that can be tolerated by a support design

The parameters that influence the rock wall deformation of the excavation include:

- Rock Type
- Field Stress
- Internal Support Pressure
- Squeezing (excessively high stress/strength ratio) conditions
- Tunnel Size
- De-stressing

The outputs that can be derived from the GRCs and results of the numerical models are:

- Expected deformation could be applied to a support design
- Support pressure requirements to ensure that an excavation remains stable in a particular geotechnical situation.
6 Assessment of shotcrete material parameters through laboratory testing

A programme of laboratory testing was undertaken to determine the characteristics of steel and polypropylene fibre reinforced shotcrete with different fibre densities. The objectives of the test programme were to:

- To improve the understanding of the support capacity provided by fibre reinforced shotcrete;
- To determine the effect of fibre density on fibre reinforced shotcrete performance;
- To obtain typical steel fibre reinforced shotcrete properties for a range of steel and polypropylene fibres;
- To compare and evaluate standard test methods for applicability in underground mines.

6.1 Test suitability

The suitability of a test can be related to a number of issues (Bernard, 1999):

- A post crack test must produce results that are structurally relevant to the intended application for the material. Relevance here should relate to the shotcrete when it is sprayed underground.
- A toughness test must be based on specimens that are representative of the shotcrete used in the finished structure. This will enable correct evaluation of results for similar sprayed shotcrete underground.
- Results obtained from the test must be a reliable indication of the material performance. This entails following the test procedures as well as ensuring that all the specimens for a given test undergo similar conditions up to the testing stage.
- A test must be economical. Economy is determined by cost of procedure, production, transportation and reliability.

6.2 Test programme

Test samples were sprayed by an experienced nozzleman on surface, under controlled conditions at the premises of BASF. The samples were sprayed, cured and tested as required by the EFNARC and ASTM beam, panel and uniaxial compressive test standards.
Eleven batches of 40MPa shotcrete were sprayed with the following fibre densities:

- Eight batches using polypropylene fibre with fibre densities of 1kg/m³ up to 8kg/m³ and
- Three batches using steel fibre having fibre densities of 40kg/m³, 55 kg/m³ and 70kg/m³.

Table 6-1 shows the test programme per fibre density batch. In total, 220 samples were tested from all eleven fibre density batches. A single fibre density batch was prepared and sprayed on a given day to ensure consistency and to avoid contamination between batches. The shotcrete was sprayed into standard EFNARC and ASTM RDP moulds. Each sample was screeded to the mould, immediately after spraying, to avoid variations in thickness. The samples were all cured in tanks under controlled temperature conditions. Each batch of samples was tested 28 days after spraying.

**Table 6-1: Test programme per fibre density batch**

<table>
<thead>
<tr>
<th>INDEX</th>
<th>DIMENSIONS (mm)</th>
<th>Number of moulds</th>
<th>Number of samples available</th>
<th>Number of samples tested</th>
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<td></td>
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<td>Width</td>
<td>Depth</td>
<td>Diameter</td>
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<td>100</td>
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<tr>
<td>*EFNARC Beams</td>
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<td></td>
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<tr>
<td>ASTM RDPs</td>
<td></td>
<td>75</td>
<td>800</td>
<td></td>
</tr>
<tr>
<td>*ASTM Beams</td>
<td>450</td>
<td>100</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>*UCS Cores</td>
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<td></td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>*Fibre density cores</td>
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<td>100</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

*Beams, Uniaxial Compressive Strength (UCS) cores and fibre density cores were cut or drilled out of the EFNARC moulds.

**6.3 Fibre specifications**

Steel fibres and polypropylene fibres were used in the tests.

**6.3.1 Steel Fibres**

The steel fibres used were hook end type as shown in the diagram in Figure 6-1 and were provided by Metalloy fibres. Table 6-2 shows the steel fibre dimensions and tensile strength.
Figure 6-1: Steel fibre dimensions

Where:
d: diameter in mm
L: fibre length in mm
I: the hook range in mm
h: the hook depth in mm

Table 6-2: Steel fibre specifications

<table>
<thead>
<tr>
<th>Fibre</th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>I (mm)</th>
<th>H (mm)</th>
<th>σ_(f) (MPa)</th>
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<td>1-2</td>
<td>1100</td>
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</tbody>
</table>

6.3.2 Polypropylene fibres

This fibre is extruded from a natural Polypropylene homo polymer and formed into a flat shape with a profiled surface in order to anchor it in a cementitious matrix (Figure 6-2). Fibres were provided by Meyco. The specifications of the fibre are listed in Table 6-3.
Figure 6-2: Polypropylene fibre

Table 6-3 Polypropylene fibre specifications

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<th>Fibre</th>
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<th>Width (mm)</th>
<th>Thickness (mm)</th>
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6.4 Standard test methods used for the determination of shotcrete performance

The standards used for the determination of fibre reinforced shotcrete performance are summarised in Table 6-4.

Table 6-4: Summary of standard test methods for testing shotcrete beams and panels

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<td>ASTM Beams</td>
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<tr>
<td>EFNARC Beams</td>
<td>EFNARC Specifications for beams (1996)</td>
</tr>
<tr>
<td>EFNARC Panels</td>
<td>EFNARC Specifications for panels (1996)</td>
</tr>
<tr>
<td>RDPs</td>
<td>ASTM C 1550 (2005)</td>
</tr>
</tbody>
</table>
6.4.1 ASTM beams

Performance was measured in terms of:

a) **Modulus of rupture (R):** is defined as the maximum surface stress in a beam at the instant of failure (first crack stress) and is given by the following formula according the ASTM C1018 (1997) standard:

\[ R = \frac{3 P L}{2 b d^2} \]

Where:

- \( R \) = modulus of rupture (N/mm²)
- \( b \) = average specimen width (mm)
- \( P \) = peak load (N),
- \( d \) = average specimen depth (mm) and
- \( L \) = span length (mm)

(b) **Toughness index:** is defined as the ratio of the absorbed energy up to a given deflection to the absorbed energy up to first crack deflection. Toughness indices i.e. \( I_5, I_{10}, I_{20}, I_{30} \) and \( I_{50} \) are calculated as specified in the ASTM C1018 (1997) Standard where:

(i) Toughness index \( I_5 \) is the number obtained by dividing the area under the load-deflection curve up to at a deflection of 3.0 times the first-crack deflection by the area up to first crack.

(ii) Toughness index \( I_{10} \) is the number obtained by dividing the area under the load-deflection curve up to a deflection of 5.5 times the first-crack deflection by the area up to first crack.

(iii) Toughness index \( I_{20} \) is the number obtained by dividing the area under the load-deflection curve up to a deflection of 10.5 times the first-crack deflection by the area up to first crack.

(iv) Toughness index \( I_{30} \) is the number obtained by dividing the area under the load-deflection curve up to a deflection of 15.5 times the first-crack deflection by the area up to first crack.
(iv) Toughness index $I_{50}$ is the number obtained by dividing the area under the load-deflection curve up to a deflection of 25.5 times the first-crack deflection by the area up to first crack.

Figure 6-3 shows graphically how the toughness indices are estimated under the ASTM load-deflection curve.

From Figure 6-3, the toughness indices are obtained by doing the following calculations:

\[
    I_5 = \frac{OACD}{OAB},
\]

\[
    I_{10} = \frac{OACEF}{OAB}
\]

\[
    I_{20} = \frac{OACEGH}{OAB}
\]

\[
    I_{30} = \frac{OACEGH}{OAB}
\]

\[
    I_{50} = \frac{OACEGIKL}{OAB}
\]

Therefore toughness index is a dimensionless quantity. For the idealised perfectly elasto-plastic material, the toughness indices have the following values:

$I_5 = 5$, $I_{10} = 10$, $I_{20} = 20$ and $I_{30} = 30$ and $I_{50} = 50$ (Papworth et al, 1996). However, for steel fibre reinforced shotcrete, Morgan (1988) performed several tests and based on the results obtained, suggested the following ratings shown in Table 6-5.

**Table 6-5: Suggested toughness indices (After Morgan, 1988)**

<table>
<thead>
<tr>
<th>Category</th>
<th>Rating</th>
<th>$I_{10}$</th>
<th>$I_{30}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Marginal</td>
<td>&lt;4</td>
<td>&lt;12</td>
</tr>
<tr>
<td>II</td>
<td>Fair</td>
<td>4</td>
<td>12</td>
</tr>
<tr>
<td>III</td>
<td>Good</td>
<td>6</td>
<td>18</td>
</tr>
<tr>
<td>IV</td>
<td>Excellent</td>
<td>8</td>
<td>24</td>
</tr>
</tbody>
</table>
6.4.2 EFNARC beams

Performance was measured in terms of:

**Flexural strength (FS)**

Flexural strength is defined as the equivalent maximum elastic tensile stress at the peak load (EFNARC, 1996). Flexural strength for EFNARC beams is calculated as specified in the EFNARC (1996) Standard using the following equation:

\[
FS = \frac{P_{0.1} \times L}{b \times d^2}
\]

Where:

- \( FS \) = Flexural strength (N/mm\(^2\))
- \( P_{0.1} \) = peak load (N)
- \( L \) = span length (mm)
- \( b \) = actual beam width
- \( d \) = actual beam depth
**Residual strength:** The residual flexural strength (RS) was determined as specified in the EFNARC (1996) Standard at displacements of 0.5mm, 1.0mm, 2.0mm and 4.0mm, as shown in Figure 6-4, using the equation below:

\[
RS = P_x \cdot L / (b \cdot d^2)
\]

Where:

- \( P_x \) = load at \( x = 0.5; 1.0; 2.0; \) and 4.0mm respectively.
- \( L \) = span length (mm)
- \( b \) = actual beam width
- \( d \) = actual beam depth

**Figure 6-4:** *P values from the Load-Deflection graph for flexural and residual strengths (EFNARC, 1996)*

**6.4.3 EFNARC Panels**

Performance was measured in terms of:

- (a) Peak loads and
- (b) Energy absorption
6.4.4 ASTM centrally loaded Round Determinate Panels

Performance was measured in terms of:

(a) Peak loads and

(b) Energy absorption

6.4.5 Uniaxial Compressive Strength (UCS) tests

UCS testing cores were drilled out of EFNARC Panels. The cores had a diameter and a length of 100mm, giving a diameter to length ratio of 1.

6.4.6 Spraying losses of steel fibres

Fibre losses were only determined for steel fibres as the laboratory used did not have the capacity to determine the polypropylene fibre losses for polypropylene fibre reinforced shotcrete specimens.

6.5 Presentation and discussion of results

The results obtained from all the tests have been presented graphically in the form of load-deflection and energy absorption-deflection curves as shown in Appendix D. The results for the performance parameters of beams and panels have been summarised and tabulated in Table 6-6. The following sub-sections present a discussion of the test results for all the fibre reinforced shotcrete tests for beams, panels and UCS tests.
<table>
<thead>
<tr>
<th>UCS</th>
<th>EFNARC BEAMS</th>
<th>ASTM BEAMS</th>
<th>EFNARC PANELS</th>
<th>ASTM ROUND PANELS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fibre content</td>
<td>UCS</td>
<td>Fibre content</td>
<td>UCS</td>
<td>Fibre content</td>
</tr>
<tr>
<td>(Kg/m³)</td>
<td>(MPa)</td>
<td>(N/mm²)</td>
<td>0.5mm</td>
<td>1.0mm</td>
</tr>
<tr>
<td>na</td>
<td>46.00</td>
<td>9.17</td>
<td>5.64</td>
<td>1.63</td>
</tr>
<tr>
<td>na</td>
<td>44.17</td>
<td>8.83</td>
<td>5.07</td>
<td>1.24</td>
</tr>
<tr>
<td>na</td>
<td>49.67</td>
<td>9.91</td>
<td>4.59</td>
<td>1.10</td>
</tr>
<tr>
<td>na</td>
<td>49.50</td>
<td>9.17</td>
<td>4.03</td>
<td>1.14</td>
</tr>
<tr>
<td>na</td>
<td>51.17</td>
<td>9.27</td>
<td>4.32</td>
<td>1.76</td>
</tr>
<tr>
<td>na</td>
<td>50.00</td>
<td>7.28</td>
<td>1.91</td>
<td>1.51</td>
</tr>
<tr>
<td>na</td>
<td>52.33</td>
<td>6.73</td>
<td>1.72</td>
<td>1.54</td>
</tr>
<tr>
<td>na</td>
<td>46.33</td>
<td>6.52</td>
<td>1.85</td>
<td>1.58</td>
</tr>
<tr>
<td>30.02</td>
<td>60.83</td>
<td>6.17</td>
<td>3.01</td>
<td>3.21</td>
</tr>
<tr>
<td>36.31</td>
<td>57.17</td>
<td>5.78</td>
<td>3.40</td>
<td>3.83</td>
</tr>
<tr>
<td>54.95</td>
<td>61.17</td>
<td>8.45</td>
<td>5.37</td>
<td>4.92</td>
</tr>
</tbody>
</table>
6.5.1 ASTM beams modulus of rupture

The modulus of rupture results in Figure 6-5 and Figure 6-6 show that fibre incorporation has little effect on the modulus of rupture of the ASTM beams. The modulus of rupture for most of the polypropylene fibre reinforced shotcrete and steel fibre reinforced shotcrete ranges between 7 and 10MPa. However, the 2kg/m³ polypropylene fibre reinforced shotcrete had a modulus of rupture less than 5MPa. This anomalous lower value could be a factor of cracking in the beam before testing. The 55kg/m³ for the steel fibre reinforced shotcrete specimens show a lower value compared to the 40Kg/m³ with a lower density. The lower modulus of rupture for the 55kg/m³ may be attributed to pump problems experienced during the spraying of the 55kg/m³ fibre reinforced shotcrete specimens.

*Figure 6-5: Modulus of rupture for the polypropylene fibre reinforced shotcrete ASTM beams*
The general trend of the toughness indices shown in Figure 6-7 and Figure 6-8 is that the toughness index value increases as the deflection increases from 3\( \delta \) to 25.5\( \delta \) (\( \delta \) = deflection at first crack) per fibre density, for both steel and polypropylene fibre reinforced shotcrete ASTM beams. However, Figure 6-7 and Figure 6-8 show that \( I_5 \) and \( I_{10} \) toughness values do not increase significantly as the fibre density in shotcrete increases. This is because \( I_5 \) and \( I_{10} \) values are calculated at very low deflections when the fibres in the shotcrete are less effective. Also, this could be a result of the scatter in the load-deflection results (shown in Appendix D), reflecting lack of repeatability of the test method in giving consistent results; or the challenge in determining the first crack, which greatly influences the toughness value calculation when using the ASTM standard test method.

\( I_{20} \) to \( I_{50} \) toughness values for steel fibre reinforced shotcrete specimens are higher than those of polypropylene fibre reinforced shotcrete specimens. The higher toughness values for the steel fibre reinforced shotcrete specimens is likely to be due to the higher tensile strength of steel fibres which is about 1100MPa compared to the 100MPa of polypropylene fibres, as indicated in Table 6-2 and Table 6-3 in section 6.3.1 and section 6.3.2 respectively.
Figure 6-7:  **Toughness indices for the polypropylene fibre reinforced shotcrete ASTM beams**

Figure 6-8:  **Toughness indices for the steel fibre reinforced shotcrete ASTM beams**

6.5.3 EFNARC beams flexural strength

Flexural strength gives an indication of the resistance to bending. The flexural strength results obtained are shown in Figure 6-9 and Figure 6-10. The polypropylene fibre reinforced shotcrete...
specimens with fibre densities ranging from 1kg/m³ to 5kg/m³ have been omitted from the discussion since they were mistakenly tested on a span length of 300mm instead of the standard 450mm for the EFNARC beams. Only the 6kg/m³ to 8kg/m³ polypropylene fibre reinforced shotcrete specimens and the steel fibre reinforced shotcrete specimens, which were tested on the correct span length of 450mm, have been incorporated in this discussion.

Figure 6-9 and Figure 6-10 show that steel fibre reinforced shotcrete has slightly higher flexural strength than polypropylene fibre reinforced shotcrete. The higher flexural strength for the steel fibre reinforced shotcrete may be attributed to the superior tensile strength of steel fibres compared to that of polypropylene fibres, as shown in Table 6-2 and Table 6-3. Also, steel fibres have a much higher modulus of elasticity of about 200GPa compared to the 1.6GPa of the polypropylene fibres used in the experiment. However, the density of fibres, whether steel or polypropylene, has a negligible effect on the shotcrete flexural strength. This is due to the fact that the flexural strength is estimated using the peak loads at very low deflections when the shotcrete matrix itself has more influence than the fibres incorporated in the shotcrete. The fibres become more “engaged” after the shotcrete experiences the first crack.

![Diagram](image)

**Figure 6-9:** Flexural strength for the polypropylene fibre reinforced EFNARC beams
6.5.4 EFNARC Beams residual strength

The residual strength gives an indication of the post peak performance of the fibre reinforced shotcrete. Figure 6-11 and Figure 6-12 show the residual strength results obtained for both steel and polypropylene fibre reinforced shotcrete EFNARC beams. The polypropylene fibre reinforced shotcrete beams which were tested on a shorter span length of 300mm rather than the standard 450mm have been omitted from this discussion for consistency purposes when using the standard test method.

Figure 6-11 and Figure 6-12 show that the steel fibre reinforced beams have a higher residual strength than the polypropylene fibre reinforced beams. The higher residual strength of steel fibre reinforced shotcrete beams may also be attributed to the higher tensile strength of steel fibres, which is about 1100MPa compared to that of polypropylene fibres of about 100MPa.

The polypropylene fibre reinforced shotcrete beams in Figure 6-11 display effectively a constant residual strength as the deflection increases from 0.5mm to 4.0mm per fibre density. This may be attributed to their ability to elongate about 24%, which results in an almost constant load capacity as deflection increases (Appendix D).

The residual strengths for steel fibre reinforced beams shown in Figure 6-12 display a much clearer linear trend which shows, as could be expected, an increase in residual strength as the fibre density increases. The magnitude of the residual strength decreases as the deflection
increases from 0.5mm to 4.0mm per fibre density. This was probably due to the steel fibres yielding, pulling out of the shotcrete, or breaking as the deflection increases thereby causing the beam capacity to decrease.

**Figure 6-11**: Residual strength for the polypropylene fibre reinforced shotcrete EFNARC beams

**Figure 6-12**: Residual strength for the steel fibre reinforced shotcrete EFNARC beams
6.5.5 EFNARC panels peak load results

The EFNARC panels peak load capacity results are shown in Figure 6-13 and Figure 6-14. The polypropylene fibre reinforced shotcrete panels peak loads range from 35 kN to 71 kN whereas those of steel fibre reinforced shotcrete panels range from 75 kN to about 100 kN. The higher tensile strength and much greater modulus of elasticity of the steel fibres compared with the polypropylene fibres again probably contributed to the higher peak strength of the steel fibre reinforced shotcrete panels.

The irregularity in the shape of the graph in Figure 6-13 may be attributed to a number of reasons. Firstly, the testing laboratory did not have the capacity to measure the true fibre content of the polypropylene specimens. Figure 6-13 reflects the specified fibre densities and not the measured fibre densities. It is possible that during the spraying process, the percentage of fibre losses may have been inconsistent resulting in lower actual fibre densities in some specimens than in others. Further explanations may include differences in panel thicknesses, as it is impossible to produce perfect standard sizes when cutting the specimens; hair line cracks formed during curing, that were noticed in some of the specimens, may have also contributed to the irregularity in the trend in Figure 6-13; lastly, some of the specimens could not be placed perfectly flat on the testing machine stand due to the slightly irregular shape produced from the cutting process.

Although there are only three results, the steel fibre reinforced shotcrete EFNARC panels in Figure 6-14 show the expected linear trend as the peak load capacity increases with the increase in fibre density.
Figure 6-13: Peak loads for the polypropylene fibre reinforced shotcrete EFNARC panels

Figure 6-14: Peak loads for the steel fibre reinforced shotcrete EFNARC panels

6.5.6 EFNARC panels energy absorption

The energy absorption results for the EFNARC panels are shown in Figure 6-15 for the polypropylene fibre reinforced shotcrete panels and in Figure 6-16 for the steel fibre reinforced shotcrete panels. Generally, both steel and polypropylene fibre reinforced shotcrete specimens
show a linear variation as the energy absorption increases due to an increase in fibre content. Therefore the more the fibres added to the shotcrete, the greater the energy absorption capability of the fibre reinforced shotcrete.

Energy absorption results in Figure 6-15 show that the 5kg/m$^3$ density and the 7kg/m$^3$ show some irregularity in the shape of the graph. As explained above, no actual fibre density measurements were done for the polypropylene fibre reinforced panels, and inconsistent fibre contents may have been a cause of the irregularity.

![Figure 6-15](image)

**Figure 6-15**  Energy absorption for the polypropylene fibre reinforced shotcrete EFNARC panels
6.5.7 ASTM Round determinate panels peak loads

The peak load results for the Round Determinate Panels (RDPs) are shown in Figure 6-17 and Figure 6-18. Figure 6-17, showing the polypropylene fibre reinforced shotcrete panels peak loads, displays a downward trend in the peak loads graph as the fibre content increases from 1 kg/m³ to 8 kg/m³. The downward trend may be attributed to the irregularities in the shapes and measured thicknesses of the panels causing a reduction in the peak load capacity as the specified fibre density in the shotcrete mix increases. The expected trend would have been a flat graph since the peak load is reached at low deflections when the fibres are not “engaged”. Since the fibres are not engaged at low deflections, the peak load capacity of the shotcrete is largely dependent on the shotcrete matrix strength as mentioned by Stacey (2004).

Although there are only three results, the steel fibre reinforced shotcrete EFNARC panels in Figure 6-18 show the expected linear trend as the peak load capacity increases with the increase in fibre density.

Figure 6-16: Energy absorption for the steel fibre reinforced shotcrete EFNARC panels
6.5.8 ASTM Round Determinate Panels (RDPs) Energy Absorption

The average energy absorption results for the RDPs are shown in Figure 6-19 and Figure 6-20 for the polypropylene and the steel fibre reinforced shotcrete panels. Results for both the steel
and polypropylene round determinate shotcrete panels show a positive linear trend. Since energy absorption has an effect on the residual carrying capacity of shotcrete, it has been found worthwhile to present the average RDP load-deflection results for both the polypropylene and steel fibre reinforced shotcrete specimens as shown in Figure 6-21 and Figure 6-22. Of major interest is the shape of the graphs post the peak load.

*Figure 6-19: Energy absorption for the polypropylene fibre reinforced round determinate panels*
Figure 6-20: Energy absorption results for the steel fibre reinforced round determinate panels

Figure 6-21 shows that after the peak load, there is a sharp drop in the load-deflection graphs to the point when crack arrest by the polypropylene fibres comes into effect. After the sharp drop, the load deflection graphs display an almost flat trend in the residual load capacities as deflection increases to 40mm deflection. The ability of polypropylene fibres to elongate to about 24% their original length may have enabled them to maintain this residual load capacity with increase in deflection.

The steel fibre reinforced RDP shotcrete panels in Figure 6-22 display a more gradual decrease in the load capacity after the peak as the deflection increases. The trend displayed by the steel fibre reinforced shotcrete RDPs could be attributed to the significant tensile properties of the steel fibres in the shotcrete which influence the load-deflection behaviour in the shotcrete.
Figure 6-21: Load-deflection results for the polypropylene fibre reinforced shotcrete round determinate shotcrete panels

Figure 6-22 Load-deflection results for the steel fibre reinforced shotcrete round determinate shotcrete panels
6.5.9 Uniaxial compressive strength

The uniaxial compressive strength (UCS) test results are presented in Figure 6-23 and Figure 6-24 for the polypropylene and steel fibre reinforced shotcrete specimens respectively.

Figure 6-23 shows that the UCS of the steel fibre reinforced shotcrete specimens ranges between 50 MPa and 65 MPa. As can be seen in Figure 6-23, the density of steel fibre has a negligible effect on the UCS of shotcrete. During the spraying of 55 kg/m$^3$ steel fibre density shotcrete, clogging problems were encountered due to pump failure. These clogging problems encountered appear to have slightly affected the UCS of one of the 55kg/m$^3$ steel fibre density shotcrete specimens.

The UCS results for polypropylene fibre reinforced shotcrete specimens range between 40 MPa and 60 MPa. An increase in polypropylene fibre density has a negligible effect on UCS as displayed in Figure 6-24.

![Figure 6-23: Uniaxial compressive strength test results for the steel fibre reinforced shotcrete cores](image-url)
6.5.10 Steel fibre spraying losses

The determination of the actual density of fibres in the final shotcrete product was carried out to give an estimate of the fibre rebound losses due to spraying (Figure 6-25). Due to the limitation in fibre determination by the testing laboratory used in the laboratory tests, only the steel fibre densities were determined for the final fibre reinforced shotcrete product. The measured results showed variation between 15 % and 30 % fibre rebound losses for the 40kg/m$^3$ and the 70kg/m$^3$ shotcrete specimens. Clogging problems caused by pump failure while spraying the 55kg/m$^3$ specimens are likely to be the cause for their increased fibre losses which ranges between 20 – 45%. However, the steel fibre rebound losses for the 40kg/m$^3$ and the 70kg/m$^3$ densities in the laboratory tests are higher than those reported by Banthia et al (1994), which were indicated to range between 12 – 18%. The difference in fibre losses may be due to the difference in operator skills or differences in the efficiencies of the mixing and spraying equipment used.
Fibre spraying losses for the samples obtained from underground (section 4) have been summarised in Table 6-7 together with the fibre losses for the shotcrete samples sprayed under controlled conditions at BASF premises. Table 6-7 shows that the average fibre losses for shotcrete applied on to the rock wall are higher than those for shotcrete sprayed into EFNARC moulds. The lower fibre losses for EFNARC moulds may be due to the EFNARC mould sides limiting the fibre rebound. The maximum and minimum fibre losses indicate the range of fibre losses for each set of samples tested. Shotcrete applied to the rock wall has the greatest variability.

Figure 6-25  Steel fibre loss results for the steel fibre reinforced shotcrete samples
Table 6-7: "Fibre content losses for shotcrete specimens sprayed underground and on surface"

<table>
<thead>
<tr>
<th>Fibre losses (%)</th>
<th>Maximum</th>
<th>Minimum</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cores drilled from a EFNARC mould (underground)</td>
<td>36</td>
<td>23</td>
<td>30</td>
</tr>
<tr>
<td>Cores drilled from rockwall underground</td>
<td>59</td>
<td>16</td>
<td>48</td>
</tr>
<tr>
<td>Cores drilled from EFNARC mould (controlled spraying)</td>
<td>44</td>
<td>16</td>
<td>26</td>
</tr>
</tbody>
</table>

6.6 Conclusions

Shotcrete properties have been determined for a number of standard tests methods and a summary of the results was presented in Table 6-6. The applicability of the various test methods has been examined and the effect of fibre content on the performance of shotcrete has been determined. The following conclusions can be drawn from this laboratory test programme:

- The ASTM and EFNARC beam tests provide limited information on the effect of fibre content on the performance of shotcrete:
  - The fibres do very little work during these tests due to the limited deflections allowed by the test methods (2.5 m for ASTM and 4 mm for EFNARC),
  - The incorporation of fibres in shotcrete has little effect on the modulus of rupture (ASTM) and this appears to be controlled by the strength of the shotcrete matrix,
  - An improvement in the values of ASTM toughness indices ($I_{20}$ to $I_{50}$) could be observed with fibre content, but the results were quite variable, making it difficult to determine a clear trend. The steel fibre reinforced shotcrete performed better than polypropylene fibre reinforced shotcrete,
  - The incorporation of fibres in shotcrete has little effect on the EFNARC flexural strength values, but the steel fibre reinforced shotcrete performed marginally better than polypropylene fibre reinforced shotcrete,
  - Polypropylene fibre content has very little influence on the EFNARC residual strength values, but the residual strength did not decrease with increasing deflection. The residual strengths for steel fibre strength displayed a much
clearer trend with increasing fibre content. However, the residual strength decreased with increasing deflection;

- The ASTM and EFNARC panel tests provide more information on the effect of fibre content on the performance of shotcrete:
  - The test methods allow 40 mm (ASTM) and 25 mm (EFNARC) deflection and the fibres are made to work,
  - Polypropylene peak loads were not influenced by fibre content in either test method, since these fibres are softer than the shotcrete matrix and do not work until the shotcrete matrix fails,
  - A clear trend of increasing peak load with increasing steel fibre content was observed, since the fibres are stiffer than the shotcrete matrix and start to work while the shotcrete matrix is failing,
  - Both polypropylene and steel fibre content have a significant influence on the energy absorption performance and clear trends can be observed with both test methods,
  - The residual strengths of polypropylene fibre reinforced shotcrete are initially much lower than for steel fibre reinforced shotcrete, but the polypropylene fibre reinforced shotcrete maintains its residual strength, while the steel fibre reinforced shotcrete loses residual strength as the deflection increases,
  - The ASTM RDP load-deflection results were less scattered than the EFNARC panels. Therefore, the RDP testing method appears to produce better results in terms of repeatability of the test method and may be the best method for use in determining the post-peak capacity of shotcrete;

- The uniaxial compressive strength of shotcrete does not show any trend with increasing fibre content;

- The actual fibre content will always be lower than the batch fibre content due to rebound losses during spraying. Fibre losses are greater when applied on the rock wall underground than during controlled spraying into moulds. Fibre losses can vary 16% to 60%.
The use of yield line analysis and test panels for the design of shotcrete

Yield Line Design is a commonly used method for the design of reinforced concrete slabs. The method was first pioneered by KW Johansen in the 1940’s and 50’s and subsequently further developed by many authors during the 1960’s to 1980’s. The method has since become well-established with structural engineers as it has economy, simplicity and versatility as its main benefits. It uses plastic yield theory (yield line theory) to investigate failure mechanisms at their ultimate limit state. It is therefore less conservative than the elastic methods of concrete design and more suitable for the design of underground mining applications of shotcrete.

This chapter concentrates on the use of the Yield Line method to establish the moment demand on the shotcrete on the wall of an excavation and to calculate the moment capacity from standard test samples to assist with the actual design of shotcrete. In the process, significant results from both an analysis/demand calculation point of view as well as insights into the behaviour/capacity of shotcrete panels under load from actual tests are reported and discussed. The method is also used to determine the moment demand and shotcrete capacities at the experimental sites (chapter 4) and provide further insights into the behaviour of shotcrete.

The chapter discusses the following aspects:

- A description of the yield line method,
- Determination of a yield line solution for shotcrete and rockbolts,
- Development of methods for determining rock loading demand,
- Development of methods for determining shotcrete capacity,
- Application of the yield line method (panel tests and underground monitoring).

7.1 Yield line Method

The Yield Line Method is based on the assumption that at the ultimate limit state, a collapse mechanism forms when the plastic properties of the reinforced concrete are reached. The collapse mechanism is defined by the designer as a pattern of yield lines (cracks) along which the reinforcement has yielded and a plastic hinge has formed. (The reader is referred to “Practical Yield Line Design” by Kennedy and Goodchild, 2003 for further reading on the application of yield line design. This document includes a number of practical examples).
7.1.1 Yield lines and collapse mechanisms

Yield lines form at the most highly stressed areas and develop into continuous plastic hinges (an example is shown in Figure 7-2 and Figure 7-3, where a load is applied to the upper surface of a simply supported square panel). They divide the concrete panel into individual regions, which pivot about their axes of rotation. One of the important aspects of yield line design is the visualisation of a set of rigid regions separated by plastic hinges along the yield lines in a particular crack pattern to form a collapse mechanism. The plastic hinge represents a crack where work is done (energy consumed) in plastic yielding of the reinforcement in comparison with the energy involved in displacing the load.

\[\text{Figure 7-1: Onset of crack formation at the point of maximum deflection on the bottom surface of a simply supported panel (modified from Kennedy and Goodchild, 2003)}\]
Figure 7-2: The formation of a mechanism in a simply supported two-way slab with the bottom steel having yielded along the yield lines (modified from Kennedy and Goodchild, 2003)

In the simply supported example, shown in Figure 7-1 and Figure 7-2, yield lines form only on the lower surface of the concrete slab. If the slab is supported in a more complex manner, yield lines may develop on both the upper and lower surfaces of the concrete slab.

In the case of shotcrete applied to a rock wall, which is also supported with rockbolts, the shotcrete panel is continuous with fixed supports (Figure 7-3). If a load is applied to the inner surface (eg. bulking of the fractured rock) cracks will develop on both the inner and outer surfaces of the shotcrete. In Figure 7-3, the yield lines on the inner surface are shown as dashed lines, while the lines on the outer surface are shown as solid lines.
The yield line is either positive or negative depending on the sense of the rotation and deflection of the concrete face (inner or outer) on which the crack/hinge manifests itself. In conventional slab design, it is convenient to depict the cracks that appear on the bottom face of the slab (towards the centre of the span) to be positive (sagging), while those on the upper face (at the supports) to be negative (hogging). The sign convention is however immaterial since the work done in forming both the positive and negative crack is additive with respect to the work done by the load as it displaces.

7.1.2 The work method

The work method is used to analyse the collapse mechanism and is based on the principle that:

\[ \text{Internal Work Done (I)} = \text{External Work Done (E)} \]

\[ \text{Work Done in Yield Line rotation} = \text{Work Done by Loads Displacing} \]

\[ \sum (W \delta) \text{ for all regions} = \sum (ML \theta) \text{ for all regions} \]

where
• $W$ is the Load acting within a particular region (kN)
• $\delta$ is the displacement of the load $W$ on each region in the direction of $W$ expressed as a fraction of unity (m)
• $M$ is the moment or moment of resistance of the slab per metre run represented by the reinforcement crossing the yield line (kNm/m)
• $L$ is the length of yield line or its projected length onto the axis of rotation for that region (m)
• $\theta$ is the rotation of the region about its axis of rotation ($\delta/R$), where $R$ is the length of the rotation arm (m) ($R=L/2$ in the example).

These parameters are illustrated in Figure 7-4.

Figure 7-4: Simply supported slab yield line pattern showing the work method parameters (after Kennedy and Goodchild, 2003)

Note that $\delta$ is on both sides of the equation and effectively cancels out. It is therefore not critical to know the amount of displacement required for failure and a unit displacement is considered in the analysis.
If a point load is applied to centre of the panel (eg. standard panel tests), $E$ is determined using the full applied load ($W$). In the case of a uniformly distributed load per square metre ($w$) (eg. bulking of fractured rock), $E$ is determined for each region as follows:

$$E = w \cdot a \cdot b \cdot \frac{L_1}{L_2} \cdot \delta$$

Where $a$ and $b$ are the length and width of the region over which the distributed load is acting.

$L_1$ is the perpendicular distance of the resultant force from the axis of rotation of the region.

$L_2$ is the perpendicular distance to the location of $\delta_{\text{max}}$ from the axis of rotation of the region.

![Diagram showing the calculation of $L_1$ and $L_2$](image)

**Figure 7-5:** Lengths $L_1$ and $L_2$

For rectangular and triangular regions, the following values of $L_1/L_2$ can be used when dealing with distributed loads:

- $1/2$ for all rectangular regions
- $1/3$ for all triangular regions with apex at point of maximum deflection
- $2/3$ for all triangular regions with apex on the axis of rotation.
When both positive and negative yield lines develop, the work done in yield line rotation for both yield lines is added for each region.

### 7.1.3 Selection of crack patterns

The designer must first postulate crack patterns and then analyse the collapse mechanisms. It is important to note that not all postulated crack patterns allow the formation of kinematically admissible collapse mechanisms. The postulated crack pattern must be such that the rotation at the hinges and the deflection of the regions can lead to failure without violating physical constraints (supports for example). Yield line design therefore demands familiarity with crack patterns and knowledge of how shotcrete panels will fail. The following rules apply to the formation of yield lines and axes of rotation, which help with the identification of valid patterns:

- Axes of rotation generally lie along lines of support and pass alongside any fixed point supports,
- Yield lines are straight,
- Yield lines between adjacent rigid regions must pass through the point of intersection of the axes of rotation of those regions,
- Only positive yield lines are developed with simple supports,
- Negative yield lines are developed along continuous supports,
- Positive yield lines develop between continuous supports,
- Yield lines propagate towards fixed supports

In theory, there may be several possible valid yield line patterns that could apply to a particular configuration of a panel and loading. However, there is one yield line pattern that gives the highest moments or least load at failure. This is known as the yield line solution. An exhaustive search is rarely necessary and selecting a few simple and obvious patterns is generally sufficient as their solutions are within a few percent of the perfectly correct solution.

It is often recommended that for design purposes, the so-called 10% rule be used to factor the design load/moment to account for any chance that the ‘true’ collapse values are bracketed.

### 7.1.4 Ultimate limit state design

According to the yield line theory, cracking and failure of a reinforced concrete slab will take place when a minimum load, for a given moment, or a maximum moment for a given load, is
reached. The load represents the Demand on the system while the moment represents the Capacity of the system. In design, the idea is to equate the Capacity and Demand in such a way that a predefined “factor of safety” is maintained. In Ultimate Limit State Design, of which Yield Line is a method, the concept of partial factors for demand and the capacity are applied so that the factored Demand and Capacity can be directly equated. The loads/demands have partial factors greater than 1.0, generally in the range 1.0 to 1.6; and the recommended partial factor for reinforced concrete in flexure is 1.5 (SABS 0100-1:1992 p9). For the Yield Line calculations, Load, w and the Moment of resistance, m are the factored or Ultimate Limit State parameters. The resulting equation gives a stable system in which \( C = D \).

In the design of shotcrete for underground purposes, the challenge is two-fold: (1) to estimate whether the strength/capacity of the shotcrete is adequate for an applied/given rock loading (the capacity problem) or (2) to estimate the rock load/demand that a given strength of shotcrete can maintain (the demand problem). In both cases, knowledge of the rock behaviour (rock loading) and the relative strength parameters of the shotcrete is required.

### 7.2 A yield line solution for shotcrete and rockbolts

The process of Yield Line analysis is to investigate all possible kinematically admissible yield line patterns and then choose the patterns that give the minimum failure moment (capacity) or the maximum load (demand). In practice, many improbable crack patterns can be discarded from practical considerations (possible shapes of yield lines), but it remains the experience of the designer to identify the most likely and optimum patterns, particularly when the support lines are not organised on a standard rectangular pattern. For the purposes of this report, only regular crack patterns are considered.

The moment of resistance is assumed to be the same for positive (\( m \)) and negative (\( m' \)) cracks. This is a reasonable assumption since, in the case of mesh reinforcement, the mesh is likely to be centrally placed, on average, while for fibre reinforced shotcrete bending in both directions will produce the same moment of resistance. For the purposes of the yield line analysis therefore, \( m = m' \).

In the case of shotcrete and bolts, the bolts represent fixed points and crack patterns will form around the bolts. Both positive and negative cracks will develop, since the shotcrete is continuous beyond the bolts. The cracks will be similar to those that form in concrete floors supported by columns (see Kennedy and Goodchild, 2003). The crack patterns for rectangular and offset bolt patterns will be different and have therefore been analysed separately.
Figure 7-6 shows the likely crack patterns for a rectangular bolt pattern:

- Envelope
- Cross
- Right angled triangle mechanism (RAT)

\[ \text{Cross} \]

\[ \text{Envelope} \]

\[ \text{Right angled triangle} \]

\[ \text{Figure 7-6: Likely yield line (crack) patterns for a rectangular bolt pattern.} \]

Figure 7-7 shows the likely crack patterns for an offset bolt pattern:

- Parallelogram
- Triangular
- Kite mechanism
Figure 7-7: Likely yield line (crack) patterns for an offset bolt pattern

It should be noted that a fan crack pattern could occur around a rock bolt if there is punching. This mechanism is common in heavily loaded reinforced concrete floor slabs around the support columns (see Kennedy and Goodchild, 2003). This is more likely to occur when the rockbolts include bearing plates on the outer surface of the shotcrete. The overall mechanism will be more complex, but this is typically accounted for in the 10% rule when designing floor slabs.

7.2.1 Moment demand for the crack mechanisms

The minimum failure moment for each of the crack patterns described above was calculated using the work method. Only a summary of the results is described in this section together with a comparison of the solutions. Expressions for the total external energy, the total internal energy, and the minimum failure moment are given in terms of $a$ and $b$ (the bolt spacing). The expression for the minimum failure moment is also given for the special case where $a = b$ i.e. a square bolt pattern. Detailed derivations of all these expressions are given in Appendix E.
Table 7-1  Minimum failure moment for the shotcrete panel crack patterns

<table>
<thead>
<tr>
<th>Crack Pattern</th>
<th>$\sum E$</th>
<th>$\sum D$</th>
<th>$M$</th>
<th>$M ,(a = b)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross</td>
<td>$\frac{wab}{3}$</td>
<td>$8M \left(\frac{a^2 + b^2}{ab}\right)$</td>
<td>$\frac{wa^2b^2}{24(a^2 + b^2)}$</td>
<td>$\frac{wa^2}{48}$</td>
</tr>
<tr>
<td>Envelope</td>
<td>$\frac{wab}{2} - \frac{wa^2}{6}$</td>
<td>$8M \left(\frac{b}{a} + 1\right)$</td>
<td>$\frac{wa^2(3b - a)}{48(a + b)}$</td>
<td>$\frac{wa^2}{48}$</td>
</tr>
<tr>
<td>Right angled triangle**</td>
<td>$\frac{w}{6}(ax + Ky + bx)$</td>
<td>$2M \left(\frac{ay + Kx + by}{xy}\right)$</td>
<td>$\frac{wxy(ax + Ky + bx)}{12(ay + Kx + by)}$</td>
<td></td>
</tr>
<tr>
<td>Kite</td>
<td>$\frac{wab}{3}$</td>
<td>$\frac{4M(4b^2 + a^2)}{ab}$</td>
<td>$\frac{wa^2b^2}{12(4b^2 + a^2)}$</td>
<td>$\frac{wa^2}{60}$</td>
</tr>
<tr>
<td>Triangle</td>
<td>$\frac{1}{6} \left[ \frac{abw}{a + \sqrt{a^2 + 4b^2}} \left( \frac{ab + a^2bw}{a + \sqrt{a^2 + 4b^2}} \right) + ab(a^2 + 4b^2) \right]$</td>
<td>$2M \left( a^2b\sqrt{a^2 + 4b^2} + 4b^2 \right)$</td>
<td>$\frac{ab(w\sqrt{a^2 + 4b^2} - 1)}{12(a + \sqrt{a^2 + 4b^2})(a^2b\sqrt{a^2 + 4b^2} + 4b^2 + a^3b + 4ab^3)}$</td>
<td></td>
</tr>
<tr>
<td>Parallelogram</td>
<td>$\frac{wab}{3}$</td>
<td>$\frac{2M}{ab} \left( 5a^2 + 4b^2 \right)$</td>
<td>$\frac{wa^2b^2}{6(5a^2 + 4b^2)}$</td>
<td>$\frac{wa^2}{54}$</td>
</tr>
</tbody>
</table>
**For the right angled triangle, the following definitions apply:**

\[
K = \sqrt{a^2 + b^2}
\]

\[
x = \frac{ab}{a + b + K}
\]

\[
y = L \sqrt{\frac{K - a}{2K}}
\]

\[
L = \sqrt{a^2 - 2ax + 2x^2}
\]

The minimum failure moment expressions in Table 7-1 were plotted for comparison purposes and the results are given in Figure 7-8. The curves are plotted for the case \(a = 2\) m. The vertical axis is normalised for different values of the load per square metre (w).

Figure 7-8: Minimum failure moment for different crack patterns for \(a = 2\) m.

Figure 7-8 shows that the envelope crack pattern is the most likely crack pattern for shotcrete and bolts. An interesting observation from the curves is the rectangular and offset bolt patterns (cross vs parallelogram and triangle vs right angled triangle) produce similar results. It does indicate that the offset bolt pattern is marginally better. The implication is that whether one
adopts a rectangular or offset bolt pattern, the capacity of the shotcrete panel in flexure will remain the same.

During the underground monitoring (see Chapter 4, section 4.4.6) the envelope crack pattern was observed with flexural failure (Figure 7-9). This confirms that is the more likely crack pattern to occur.

![Envelope pattern formed as a combination of primary and secondary cracks](image)

**Figure 7-9: Envelope pattern formed as a combination of primary and secondary cracks**

### 7.3 Methods of determining rock loading demand

Once the critical yield line pattern has been established, the demand loading from the rock needs to be assessed. Quasi static loading, deadweight loading and dynamic loading are discussed in this section.

#### 7.3.1 Quasi-static loading

In chapter 4, section 4.5, it was shown that deformation occurs due to fracturing and flexural failure is generally due to deformation within the near zone (within the length of the bolts). Ideally, the near zone deformation should be measured or modelled, but since more fracturing occurs near the surface of the excavation, it is reasonably conservative to assume that all of the deformation takes place in the near zone.
The quasi-static loading demand can be determined from ground reaction curves or the maximum displacement.

**Ground reaction curves**

The demand placed on a support system under quasi-static loading is best determined by calculating a ground reaction curve for a tunnel in a given rock mass and stress environment (Brown, 1999, Hoek, 1998 and Papworth, 2002) (see Chapter 2). This curve represents the deformation ($\delta$) of the tunnel in response to reducing support pressure ($\nu$). Initially the high support pressures represent the confinement provided by the rock mass, but the lower support pressures represent the pressure provided by a support system. The ground reaction curves can be compared with the capacity of the support system from the time of installation as shown in Figure 7-10. The determination of shotcrete capacity is described in section 7.4.

![Ground Reaction Curves](image)

**Figure 7-10: Schematic ground reaction curves and support interaction using shotcrete panel test results (Papworth 2002)**

Hoek (1998) used the following equation to determine the ground reaction curve, which is appropriate for tunnels with a circular cross-section in a hydrostatic stress field and a uniformly applied support pressure:

$$\frac{\delta_i}{a_o} = \left( 0.002 - 0.0025 \frac{p_i}{p_o} \right) \sigma_{cm} \left( 2.4 \frac{p_i}{p_o} - 2 \right)$$
where

\[ \delta_l = \text{Tunnel sidewall deformation} \]

\[ d_o = \text{Original tunnel radius} \]

\[ p_i = \text{Internal support pressure} \]

\[ p_o = \text{In situ stress} \]

\[ \sigma_{cm} = \text{Rock mass strength} \]

Numerical modelling can be used to derive ground reaction curves for excavations with rectangular or irregular cross-sections (e.g., Leach 1994, 1995 and 1998 and Speers and Spearing, 1996). In chapter 5, ground reaction curves were determined from non-linear, distinct element (UDEC) modelling of tunnels in various rock masses, characterised by rock type and geological strength index (GSI). The rock mass was modelled using a voronoi tessellation network of joints with the rock mass strength properties. These models realistically represent the formation of extension fractures and bulking that occurs in tunnels in overstressed brittle rock underground.

**Maximum displacement**

In deep mines, the rock deformation is generally considered irresistible under quasi-static loading, since the support pressures required to stop deformation are much greater than can be practically achieved using bolts and thin layers of shotcrete. The function of shotcrete, in this case, is simply to prevent unravelling of the broken rock in between tendons and allow the rock to support itself. The shotcrete will bend in response to the deformation of the rock and will develop a resisting support pressure. This displacement will effectively use up some of the remaining support capacity, which is necessary for holding the broken rock in place under deadweight loading (section 7.3.2) and dynamic loading (section 7.3.3). If the deflection of the shotcrete is excessive, the moments developed within the shotcrete will cause failure. Small rock displacements will not develop excessive moment within the shotcrete and the shotcrete will have a high remaining capacity. The displacement capacity of shotcrete is discussed in section 7.4.

The most reliable way of determining the maximum displacement is from underground monitoring. Ideally, these should be extensometer measurements to determine the near zone displacements, but simple closure measurements could also be used.
For design purposes it is necessary to estimate the maximum expected deformation in advance and this can be done through numerical modelling. The modelled maximum displacements as a result of stress changes in 3.5 m x 3.5 m tunnels have been summarised in Table 5-8, Chapter 5. It should be noted that these are overall displacements, but as stated above, it is reasonably conservative to use these displacements as near zone displacements. Corrections can be made for tunnel size.

### 7.3.2 Deadweight loading

In deadweight loading, the weight (W) of the largest wedge that can be formed represents the rock load demand. Barrett and McCreath (1995) propose calculating the weight of a roof prism defined by the tendon spacing and joint angles of 30° to the tendon (Figure 7-9). In a very competent rock mass with few joints, this will be a conservative estimate of the rock loading demand. Adhesion loss is assumed, which will not always be the case. The clamping stresses and interacting of rock blocks (Mason and Stacey 2009) are ignored in this calculation, since they are difficult to quantify reliably. This represents a worst case scenario.
Figure 7-11  Estimating the maximum deadweight load (roof prism)

Weight of largest wedge: \[ W = \frac{\rho g ab b/2 \tan 60}{3} \]

\[ W = \frac{\rho g ab b^2}{2\sqrt{3}} \]

where:

- \( a, b \) are the larger and smaller tendon spacings respectively,
- \( \rho \) is the density of the rock, and
- \( g \) is the gravitational acceleration (9.8 m/s\(^2\))

Rock load demand:

\[ w = \frac{W}{ab} = \frac{\rho gb}{2\sqrt{3}} \]

The moment demand for the envelope pattern (Table 7-1) is therefore:
Assuming a square pattern (a=b):

\[ m_d = \frac{\rho g a^2 b (3b - a)}{96\sqrt{3}(a + b)} \]

\[ m_d = \frac{\rho g a^3}{96\sqrt{3}} \]

### 7.3.3 Dynamic loading

For dynamic loading, the roof prism analogy (Figure 7-11) in section 7.3.2 can be applied to dynamic loading. The prism can represent the largest rock wedge formed between tendons or the zone of the broken rock mass that falls outside the tendon support zone. The assumption of a 30° tendon zone of influence for broken rock is conservative. Both a roof prism and an equivalent sidewall prism can be considered.

The energy of an equivalent ejected prism is taken to be the dynamic loading demand. In the case of a roof prism, both kinetic and potential energy should be considered, while for a sidewall prism only the kinetic energy should be considered.

**Roof prism dynamic loading demand:**

\[ E = \frac{\rho a b^2 v^2}{4\sqrt{3}} + \frac{\rho g a b^2 \delta_e}{2\sqrt{3}} \]

**Sidewall prism dynamic loading demand:**

\[ E = \frac{\rho a b^2 v^2}{4\sqrt{3}} \]

where:

- a and b are the larger and smaller tendon spacings respectively
- \( \rho \) is the density of the rock
- g is the gravitational acceleration (9.8 m/s\(^2\))
- v is the ejection velocity
- \( \delta_e \) is the displacement undergone before being arrested by the shotcrete

The displacement can be estimated by considering the remaining displacement capacity of the shotcrete (see section 7.3.1 and 7.4.2).
7.4 Methods of determining shotcrete capacity

It must be noted that flexural failure is only possible if adhesion is lost (Barrett and McCreath, 1995). The Barrett and McCreath method of calculating adhesive failure is described in section 2.7.1, which is important to use when considering deadweight demand. However, during underground monitoring it was observed that adhesion is invariably lost due to the fractured nature of the rock in quasi-static loading (section 4.4.6).

Numerous authors have considered the moment capacity for the bending of shotcrete reinforced with mesh or fibre (Kirsten, 1992, Barrett and McCreath, 1995, Vandewalle, 1997 and Leach and Naidoo, 2001). These approaches are described in detail in section 2.7.7 and are not repeated here.

This section explains how yield line theory can be used to determine both the peak and the residual moment capacity of shotcrete from test panels. The determination of the energy absorption capacity from test panels is also described.

7.4.1 Moment Capacity

Often, shotcrete beam tests are available or produced and tested especially for a project. Standard testing methods such EFNARC beams are used for this purpose. Malmgrem (2001) has postulated the relationship between the moment capacity of an EFNARC beam and a shotcrete panel as:

\[
m_{\text{plate}} = \left( \frac{h_p}{h_b} \right)^2 \left( \frac{1}{b_b} \right) M_{\text{beam}}
\]

where:

- \( h_p \) is the panel thickness (75mm)
- \( h_b, b_b \) are the beam thickness (75mm) and breadth (125mm)

and

\[
M_{\text{beam}} = \frac{L W_b}{6}
\]

is the beam moment for a total “third point” peak load \( W_b \) and a span of \( L \) (450mm)

This treatment is presented as a relationship between the angular deformation and the moment of the EFNARC beam, which varies for the type and quantity of fibres used in the mix or on the mesh used. It is however proposed that the peak moment from actual tests be used and the
moment capacity for design purposes, in which case $m_c$ is derived directly from $M_{beam}$ (tests) correcting for the thickness as follows:

$$m_c = \left( \frac{h}{h_b} \right)^2 \left( \frac{1}{b_b} \right) M_b = \frac{h^2 0.45 W_b}{0.075^2 \times 0.125} = 107h^2 W_b$$

Peak moment capacity can also be derived directly from panel test results.

The peak moment capacity for a centrally loaded, square panel, simply supported on all sides (EFNARC), has been derived using yield line theory assuming the crack pattern in Figure 7-12.:

$$m_{pe} = \frac{W_{pe}}{8}$$

where:

$W_{pe}$ is the peak load (kN)

Note that the actual crack patterns are quite erratic and the value of obtained during EFNARC panel tests $W_{pe}$ are highly variable (see section 7.5.1).

To determine the moment capacity for the shotcrete applied underground, it is necessary to correct for $h_p$ (75mm) as follows:

$$m_c = \left( \frac{h}{h_{pe}} \right)^2 m_{pe} = \frac{h^2 W_{pe}}{0.075^2} = \frac{h^2 W_{pe}}{0.045}$$

*Figure 7-12: EFNARC Square Panel*
The peak moment capacity for a centrally loaded round panel (ASTM C1550), also known as a round determinate panel (RDP) (Figure 7-13), has been derived using yield line theory for the determinate crack pattern:

\[ m_{pc} = \frac{W_{pc}}{5.54} \]

The correction for 75 mm thickness is as follows:

\[ m_c = \left( \frac{h}{h_{pc}} \right)^2 m_{pc} = \frac{W_{pc}}{5.54} \times \frac{h^2}{0.075^2} = \frac{h^3 W_{pc}}{0.0312} \]

Note that the crack patterns are very consistent for RDP tests and the values of \( W_{pc} \) are quite consistent and it is therefore better to use RDP test results for design (see section 7.5.1).

**Figure 7-13: ASTM C1550 Round Determinate Panel**

The moment rotation response can be determined for a given shotcrete mix as shown in Figure 7-14. The rotation (\( \theta \)) is simply determined as follows:

\[ \theta = \frac{\delta}{R} \]

Where:

\( \delta \) is the deflection, and

\( R \) is the moment arm.
This approach is commonly used to characterize the residual response of beams and panels (Thompson, 2006, Bernard et. al 2000, Armelin and Banthia, 1997).

![Graph showing moment rotation response for Plastic and Steel Fibre reinforced shotcrete](image)

**Figure 7-14: Moment rotation response for Plastic and Steel Fibre reinforced shotcrete (after Thompson 2006)**

The moment arm for an EFNARC panel is 0.5 x the panel width of 750 mm and for an ASTM C1550 round panel is 0.5 x the spacing between supports (750 mm). For applied shotcrete the moment arm is 0.5 x the lesser of the two support spacings (b). Therefore the displacement capacity ($\delta_c$) in metres for quasi-static loading (section 7.3.1) is simply determined as follows for both EFNARC and ASTM C1550 round panels:

$$\delta_c = \frac{\delta_p b}{0.75}$$

If the maximum expected displacement exceeds the displacement capacity then the shotcrete will enter the residual state. In section 4.4.5, it was shown that the rate of displacement increased, when the shotcrete residual capacity was reduced (formation of a slab). Therefore increasing the residual capacity of the shotcrete will reduce the rate of displacement and prolong the life of the excavation, or reduce the time until rehabilitation is required.
7.4.2 Energy Absorption Capacity

The Energy Absorption capacity of applied shotcrete can also be determined from test panels using the moment-rotation response (section 7.4.1) for a given shotcrete mix. The Energy absorption is simply the area under the load (W) deflection (δ) graph.

The load bearing capacity of applied shotcrete with a rectangular tendon pattern can be determined from EFNARC $W_{pe}$ as follows (section 7.4.1):

$$W = 6W_{pe} \left( \frac{h}{0.075} \right)^2$$

The energy absorption of the applied shotcrete with a rectangular pattern can then be determined taking the moment arm into consideration (section 7.4.1):

$$EA = 6EA_{pe} \left( \frac{h}{0.075} \right)^2 \frac{b}{0.75}$$

where

$EA_{pe}$ is the EFNARC panel energy absorption

$h$ is the thickness of the applied shotcrete

$b$ is the lesser tendon spacing

The EFNARC test is limited to 25 mm of deformation and this must be considered the limit, since no further testing is done. Referring to section 7.4.1, the displacement (δ$_e$) required for the energy absorption demand (section 7.3.3) can be determined from the remaining displacement capacity of the shotcrete and the quasi-static displacement demand (section 7.3.1) as follows:

$$\delta_e = 0.025 \frac{b}{0.75} - \delta_d$$

$$\delta_e = 0.033 b - \delta_d$$

The load bearing capacity of applied shotcrete with a rectangular tendon pattern can be determined from ASTM C1550 round panel $W_{pc}$ as follows (section 7.4.1):

$$W = 8.7W_{pc} \left( \frac{h}{0.075} \right)^2$$

The energy absorption of the applied shotcrete with a rectangular pattern can then be determined taking the moment arm into consideration:

$$EA = 8.7EA_{pc} \left( \frac{h}{0.075} \right)^2 \frac{b}{0.75}$$
where

\[ EA_{pc} \] is the ASTM C1550 round panel energy absorption

The ASTM C1550 round panel test is limited to 40 mm and using the same approach, \( \delta_e \) is estimated as follows:

\[
\delta_e = 0.040 \frac{b}{0.75} - \delta_d \\
\delta_e = 0.053 b - \delta_d
\]

7.5 Application of the yield line method to the design of shotcrete (panel test and underground monitoring results)

In this section standard panel test results (section 6) are analysed to determine their suitability for design and determine typical design values. Then the underground site characteristics and monitoring results (section 4) are used to investigate the applicability of the method.

7.5.1 Yield line analysis applied to EFNARC and ASTM C1550 test panels

An extensive laboratory testing programme (section 6) was conducted as part of the research project. This comprised eight polypolyethylene fibre reinforced shotcrete and three steel fibre reinforced batches, which were sprayed on surface. For each batch, four EFNARC panels and four ASTM C1550/RDP panels were sprayed. Additional panels were also sprayed for beam and UCS tests, but these are not discussed here. The control of panel thickness was good, since all panels were screeded to the mould.

Figure 7-15 represents an example of the results, which is from the 70kg/m³ steel fibre batch. The graph shows a significant variation in both the peak load and deflection at peak load. Non-linear behaviour is evident from 40 kN, indicating that cracks are already evident. There are load losses associated with crack formation and load gains as the fibres are stretched. The peak loads range from 85.2 kN to 131.2 kN, with an average of 103.3 kN. This equates to an \( m_{pe} \) of 12.9 kNm/m. The scatter in values is quite large. The residual strength values are more consistent, with the exception of a single test (D), which has a considerably higher residual...
value. Each EFNARC test panel has a unique crack pattern, which renders the yield line determination of $m_{pe}$ using a simple rectangular crack pattern less reliable (Figure 7-12).

The deflection at peak load for these results varies from 4.1 mm to 6.9 mm, with an average of 5.4 mm. These results are highly variable.

**Figure 7-15: Example of EFNARC test results for steel fibre reinforced shotcrete (70 kg/m$^3$)**

The example of ASTM C1550/RDP tests in Figure 7-16 is also for the 70kg/m$^3$ batch (same batch as EFNARC tests, sprayed on the same day and cured in the same manner). The Peak loads of the ASTM C1550/RDP tests are far more consistent. These range from 38.9 kN to 47.7 kN, with an average of 43.0 kN. The resulting $m_{pc}$ is 7.8 kNm/m, which is approximately 0.6 of the equivalent EFNARC value. These results should be similar, since the test panels are the same thickness. The determinate pattern, which is part of the ASTM C1550/RDP test design, results in a far more consistent peak load. Also, since the crack pattern is determinate, the yield line solution is more reliable. The residual strength values are remarkably consistent.

The deflections at peak load are between 6 mm and 9 mm, which are also variable and larger than the EFNARC results.
Figure 7-16:  Example of ASTM C1550 RDP test results for steel fibre reinforced shotcrete (70 kg/m³)

Figure 7-17 represents an example of ASTN C1550/RDP test results with a polypropylene fibre batch (6 kg/m³), which is typical of the polypropylene fibre results. The peak load values are quite variable, ranging from 29 kN to 42 kN, with an average of 33 kN, giving an m_pc value of 6.0 kNm/m. However, the deflection at peak load is extremely variable, ranging from 2 mm to 4 mm. The deflections are also significantly lower than for steel fibre, implying lower quasi-static deformation capacity. The residual values are however, remarkably consistent and level. Since the modulus of polypropylene fibre is considerably lower than steel, it probably plays no part in the behaviour until peak load. This behaviour is probably due to the strength of the shotcrete without reinforcement, which is highly variable. The residual behaviour is controlled by the fibres and is therefore consistent.
Figure 7-17: Example of ASTM C1550/RDP test results for polypropylene fibre reinforced shotcrete (6 kg/m³).

The moment-crack rotation (ASTM C1550 round panel) for each of the shotcrete batches (median values) tested is shown in Figure 7-18. Note that for increasing quantities of polypropylene fibres there is no trend for the erratic peak values, but there is a clear trend observed for the consistent residual values. There is a clear trend in peak and residual values for the steel fibres. The improvement in displacement capacity with steel fibre reinforcement is evident.

The polypropylene fibre reinforced shotcrete performs better at the maximum test displacement. The steel fibres are stiffer and perform better early on, but also start failing earlier and therefore the capacity is reduced as the deflection increases. This indicates that the polypropylene fibre reinforced shotcrete can accommodate greater displacement.

Based on these results, it is expected that when applied underground, polypropylene fibre reinforcement will deform faster than steel fibre reinforced shotcrete and take less time to reach 0.107 radians, but then it will still have residual capacity and will continue to perform its function.

The choice of product depends on the service requirements underground.
The Energy Absorption capacity of shotcrete panels is calculated from the area under the load-deflection curve. If the shotcrete panel has already undergone any deflection, the remaining Energy Absorption capacity is the remaining area under the curve, beyond this deflection. The Remaining Energy Absorption capacity versus deflection determined from ASTM C1550 round panels is shown in Figure 7-19. It is interesting to note that steel fibre reinforced shotcrete has higher Energy Absorption capacity than polypropylene fibre reinforced shotcrete initially, but after undergoing more than 15 mm deflection, the situation is reversed.

Figure 7-18: Moment-rotation graphs (ASTM C1550 round panel) for steel and polypropylene fibre reinforced shotcrete batches (median values)
Figure 7-19: Remaining Energy Absorption capacity versus deflection graphs (ASTM C1550 round panel) for steel and polypropylene fibre reinforced shotcrete batches (median values)

A summary of all the laboratory test results for the surface testing programme is presented in Table 2. The number of tests (N) is indicated and the values are the 20 percentile values of those tests. Due to the variability in performance, it is appropriate to use 20 percentile values for design, since at least 80% of the shotcrete will perform better. When the data are well constrained and consistent, the 20 percentile values are close to the mean values, but are significantly lower when the data are not well constrained. Average fibre densities obtained from crushed cores of steel fibre reinforced shotcrete are shown, since considerable fibre loss can be expected. Unfortunately, it was not possible to extract the polypropylene fibres from the crushed core. Steel fibre losses are high, ranging from 22% to 34% under these controlled surface spraying conditions.

The polypropylene fibre reinforced shotcrete peak $W_{pe}$, $W_{pc}$, $M_{pe}$ and $M_{pc}$ values are extremely variable and there is no trend with increasing fibre content. They are higher than steel fibre reinforced shotcrete with ASTM C1550/RDP panel tests and the opposite is true for the EFNARC panels. This is somewhat surprising and the explanation for this is not obvious. These values should be used with caution and it is suggested that the lowest polypropylene values should be used for design purposes.
The steel fibre reinforced shotcrete $W_{pe}$, $W_{pc}$, $M_{pe}$ and $M_{pc}$ values for both EFNARC and ASTM C1550/RDP panels show a clear trend with increasing fibre content. This indicates that the steel fibres play a significant role in controlling the behaviour up to peak load. These results are more consistent and these 20 percentile values can be used.

The Energy Absorption (EA) clearly increases with increasing fibre content for both polypropylene and steel fibre reinforced shotcrete. The 20 percentile values can be used for design purposes. The high content polypropylene fibre panels are equivalent to the steel fibre panels. Note that the maximum deflections for EFNARC and ASTM C1550 panels are 25 mm and 40 mm respectively. If the test result shows a reasonable residual capacity at maximum test deflection then the calculated EA will always be conservative.

The deflection at peak load ($\delta_p$) is quite variable and very small. A trend does emerge when looking at the steel fibre panels and with a high fibre density. It is interesting that the polypropylene fibre ERFNAC panels reached peak load at extremely low deflections. This indicates that under quasi-static loading very little displacement can be accommodated before the shotcrete enters a residual state.

**Table 7-2: Summary of 20 percentile shotcrete properties determined from laboratory test results**

<table>
<thead>
<tr>
<th>Fibre reinforced shotcrete batch</th>
<th>Actual fibre content (kg/m³)</th>
<th>EFNARC</th>
<th></th>
<th></th>
<th></th>
<th>ASTM1550/RDP</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>$W_{pe}$ (kN)</td>
<td>$m_{pe}$ (kNm/m)</td>
<td>$\delta_p$ (mm)</td>
<td>EA (J)</td>
<td>N</td>
<td>$W_{pc}$ (kN)</td>
</tr>
<tr>
<td>Polypropylene 1 kg/m³</td>
<td></td>
<td>4</td>
<td>53</td>
<td>6.6</td>
<td>0.5</td>
<td>165</td>
<td>3</td>
<td>40</td>
</tr>
<tr>
<td>Polypropylene 2 kg/m³</td>
<td></td>
<td>4</td>
<td>47</td>
<td>5.9</td>
<td>0.6</td>
<td>294</td>
<td>3</td>
<td>33</td>
</tr>
<tr>
<td>Polypropylene 3 kg/m³</td>
<td></td>
<td>3</td>
<td>29</td>
<td>3.6</td>
<td>0.6</td>
<td>340</td>
<td>3</td>
<td>50</td>
</tr>
<tr>
<td>Polypropylene 4 kg/m³</td>
<td></td>
<td>4</td>
<td>46</td>
<td>5.8</td>
<td>0.6</td>
<td>412</td>
<td>3</td>
<td>33</td>
</tr>
<tr>
<td>Polypropylene 5 kg/m³</td>
<td></td>
<td>4</td>
<td>29</td>
<td>3.6</td>
<td>1.2</td>
<td>368</td>
<td>3</td>
<td>29</td>
</tr>
<tr>
<td>Polypropylene 6 kg/m³</td>
<td></td>
<td>4</td>
<td>39</td>
<td>4.9</td>
<td>0.8</td>
<td>582</td>
<td>4</td>
<td>33</td>
</tr>
<tr>
<td>Polypropylene 7 kg/m³</td>
<td></td>
<td>4</td>
<td>46</td>
<td>5.8</td>
<td>6.6</td>
<td>870</td>
<td>4</td>
<td>25</td>
</tr>
<tr>
<td>Polypropylene 8 kg/m³</td>
<td></td>
<td>4</td>
<td>60</td>
<td>7.5</td>
<td>4.2</td>
<td>1058</td>
<td>3</td>
<td>29</td>
</tr>
<tr>
<td>Steel 40 kg/m³</td>
<td></td>
<td>30</td>
<td>4</td>
<td>57</td>
<td>7.1</td>
<td>426</td>
<td>2</td>
<td>25</td>
</tr>
<tr>
<td>Steel 55 kg/m³</td>
<td></td>
<td>36</td>
<td>4</td>
<td>64</td>
<td>8.0</td>
<td>823</td>
<td>2</td>
<td>28</td>
</tr>
<tr>
<td>Steel 70 kg/m³</td>
<td></td>
<td>55</td>
<td>4</td>
<td>86</td>
<td>10.8</td>
<td>999</td>
<td>4</td>
<td>40</td>
</tr>
</tbody>
</table>
7.5.2 Yield line analysis applied to the underground monitoring results

The underground monitoring is reported in Section 4. The site characteristics and monitoring results were used to estimate the demand and capacity at the underground site and verify the design procedures.

During the underground monitoring programme EFNARC test panels were sprayed during site establishment and these were used to determine shotcrete properties on site. The underground panels were not screeded and the panel thickness was invariably greater than 75 mm and the peak loads and energy absorption were therefore corrected for the thickness. Difficulties in spraying and transporting the panels limited the number of tests. Two ASTM C1150/RDP panels were sprayed underground, but could not be tested as they were too thick. Several underground test panels were damaged and could not be tested.

At South Deep site 2 cores were drilled from shotcrete on the wall, while the other cores were obtained from core trays. Measured fibre losses are higher in the underground environment than under the controlled surface spraying conditions. Up to 50% fibre loss was measured.

The results of the laboratory testing for EFNARC panels sprayed and cured at the underground sites are presented in Table 7-3. Note that $\delta_p$ is very low for the unreinforced shotcrete at Impala.

Table 7-3: Laboratory test results for EFNARC panels sprayed and cured at underground sites

<table>
<thead>
<tr>
<th>Underground site laboratory testing</th>
<th>Actual fibre content (kg/m³)</th>
<th>N</th>
<th>$W_{pe}$ (kN)</th>
<th>$m_{pe}$ (kNm/m)</th>
<th>$\delta_p$ (mm)</th>
<th>EA (J)</th>
<th>UCS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>South Deep site 1</td>
<td>19</td>
<td>1</td>
<td>60</td>
<td>7.4</td>
<td>3.0</td>
<td>889</td>
<td>29</td>
</tr>
<tr>
<td>South Deep site 2</td>
<td>18</td>
<td>1</td>
<td>21</td>
<td>2.6</td>
<td>3.7</td>
<td>393</td>
<td>36</td>
</tr>
<tr>
<td>Mponeng 109 level</td>
<td>44</td>
<td>1</td>
<td>41</td>
<td>5.2</td>
<td>5.0</td>
<td>512</td>
<td>23</td>
</tr>
<tr>
<td>Impala</td>
<td>0</td>
<td>2</td>
<td>18</td>
<td>2.3</td>
<td>1.3</td>
<td>62</td>
<td>26</td>
</tr>
</tbody>
</table>

Several generic properties for shotcrete and reinforcement were used in this assessment and these are listed in Table 7-4. The sources of information are also listed.

The $W_{pc}$ values from the surface prepared laboratory testing programme for 40 kg/m³ and 55 kg/m³ are used to represent the underground equivalent.
For comparison, flexural capacity was also determined using Kirsten’s (1992) method described in section 2.7.7. The steel fibre strength is determined from the Harex fibre specification, which is used in the South Deep mix. It is relatively low, when compared with Dramix fibres and is possibly conservative for the Mponeng results. A typical steel density is assumed for the fibres. The mesh characteristics are typical of that used in South African mines.

**Table 7-4: Generic properties for shotcrete and reinforcement**

<table>
<thead>
<tr>
<th>Properties</th>
<th>Source</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shotcrete (W_{pc} (40\text{kg/m}^3)) (kN)</td>
<td>ASTM C1550/RDP panel test results</td>
<td>25.7</td>
</tr>
<tr>
<td>Shotcrete (W_{pc} (55\text{kg/m}^3)) (kN)</td>
<td>ASTM C1550/RDP panel test results</td>
<td>29.9</td>
</tr>
<tr>
<td>Shotcrete (\sigma_t) (MPa)</td>
<td>Beam tests</td>
<td>8</td>
</tr>
<tr>
<td>Fibre strength (MPa)</td>
<td>Harex SF01-32 specification</td>
<td>800</td>
</tr>
<tr>
<td>Steel density (kg/m(^3))</td>
<td>Typical steel density</td>
<td>7800</td>
</tr>
<tr>
<td>Mesh aperture (mm)</td>
<td>Commonly used mesh</td>
<td>50</td>
</tr>
<tr>
<td>Mesh gauge (mm)</td>
<td>Commonly used mesh</td>
<td>3.15</td>
</tr>
<tr>
<td>Mesh strength (MPa)</td>
<td>High yield steel</td>
<td>450</td>
</tr>
</tbody>
</table>

Three of the underground monitoring sites were used in the assessment; South Deep site 2, Mponeng 116 level and Impala 14 shaft. The site characteristics are listed in Table 7-5. These include the monitoring results (modelled stress and measured displacement), rock mass characteristics and support characteristics. Values in italics are estimated.

The South Deep site 2 is a pillar and the modelled \(\sigma_v\) is in fact the average pillar stress. The early and late displacement characteristics are recorded. The early displacement is taken at an observed increase in the rate of displacement. The late displacement represents an advanced state of failure in the shotcrete, where the cracks had connected to form a slab (secondary failure) and the rate of displacement again increased. Near and far displacements were determined using the MPBX results (section 4.4.1). Some of the displacement and cracking is induced by strong ground motion associated with nearby longhole blasting. The maximum PPV values were measured directly at South Deep. The late maximum PPV is considered to be a site response to longhole blasting, but is only estimated, since the instruments actually recorded much higher values, but the validity of these measurements is queried (discussed in section 4.4.3).
The displacements recorded at Mponeng and Impala are the final recorded displacements. Only minor cracking was recorded at Mponeng, which has also been subjected to ground motions from remote seismicity. The maximum PPV is calculated from the magnitude and distance of a large seismic event (section 4.4.3). No cracks were visible at the Impala site and no significant ground motions were experienced at Impala.

Shotcrete was already applied at Mponeng and the neither the rock mass characteristics nor the shotcrete properties were determined for this site. Rock mass characteristics at Mponeng 116 level are assumed using typical quartzite rock mass characteristics observed at Mponeng. It was not possible to drill cores at the site to obtain fibre densities and therefore the properties determined at the Mponeng 109 level were used, which has the same shotcrete mix. The mesh installed at 116 level is over the shotcrete and therefore does not reinforce it, but acts as additional containment if the shotcrete should fail. The mesh characteristics on site are used to provide an indication of the additional moment of resistance that could be provided with this mesh.
Table 7-5: Underground monitoring site characteristics

<table>
<thead>
<tr>
<th>Site Characteristics</th>
<th>South Deep Site 2 Early</th>
<th>South Deep Site 2 Late</th>
<th>Mponeng 116 level</th>
<th>Impala</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Monitoring results</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial Vertical stress $\sigma_v$ (MPa)</td>
<td>70</td>
<td>70</td>
<td>102</td>
<td>90</td>
</tr>
<tr>
<td>Initial Horizontal stress $\sigma_h$ (MPa)</td>
<td>5</td>
<td>5</td>
<td>61</td>
<td>120</td>
</tr>
<tr>
<td>Initial Max Tangential stress $\sigma_\theta$ (MPa)</td>
<td>205</td>
<td>205</td>
<td>245</td>
<td>150</td>
</tr>
<tr>
<td>Final Vertical stress $\sigma_v$ (MPa)</td>
<td>70</td>
<td>85</td>
<td>102</td>
<td>97</td>
</tr>
<tr>
<td>Final Horizontal stress $\sigma_h$ (MPa)</td>
<td>5</td>
<td>5</td>
<td>56</td>
<td>116</td>
</tr>
<tr>
<td>Final Max Tangential stress $\sigma_\theta$ (MPa)</td>
<td>205</td>
<td>250</td>
<td>250</td>
<td>175</td>
</tr>
<tr>
<td>Measured displacement $\delta_t$ (mm)</td>
<td>2</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Measured displacement $\delta_h$ (mm)</td>
<td>3</td>
<td>22</td>
<td>1.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Max measured PPV (mm/s)</td>
<td>200</td>
<td>1000*</td>
<td>552</td>
<td>0</td>
</tr>
<tr>
<td>Condition of shotcrete</td>
<td>Primary</td>
<td>Secondary</td>
<td>Minor</td>
<td>No cracks</td>
</tr>
<tr>
<td><strong>Rock mass characteristics</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock unit weight $\gamma$ (kN/m$^3$)</td>
<td>27</td>
<td>27</td>
<td>29</td>
<td>28</td>
</tr>
<tr>
<td>Rock strength $\sigma_c$ (MPa)</td>
<td>200</td>
<td>200</td>
<td>180</td>
<td>100</td>
</tr>
<tr>
<td>GSI</td>
<td>70</td>
<td>70</td>
<td>70</td>
<td>75</td>
</tr>
<tr>
<td>Span</td>
<td>6</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
</tr>
<tr>
<td><strong>Tendon support</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tendon spacing a (m)</td>
<td>1.5</td>
<td>1.5</td>
<td>1.4</td>
<td>1.0</td>
</tr>
<tr>
<td>Tendon spacing b (m)</td>
<td>1.5</td>
<td>1.5</td>
<td>1.4</td>
<td>1.0</td>
</tr>
<tr>
<td>Tendon Pattern</td>
<td>Square</td>
<td>Square</td>
<td>Diamond</td>
<td>Square</td>
</tr>
<tr>
<td><strong>Shotcrete characteristics</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shotcrete thickness $h$ (mm)</td>
<td>50</td>
<td>50</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>Shotcrete $\sigma_c$ (MPa)</td>
<td>36</td>
<td>36</td>
<td>30</td>
<td>26</td>
</tr>
<tr>
<td>EFNARC Peak load $W_{pe}$ (kN)</td>
<td>21</td>
<td>21</td>
<td>41</td>
<td>18</td>
</tr>
<tr>
<td>EFNARC Deflection $\delta_{pe}$ (mm)</td>
<td>3.7</td>
<td>3.7</td>
<td>5</td>
<td>1.25</td>
</tr>
<tr>
<td>EFNARC Energy absorption (J)</td>
<td>393</td>
<td>393</td>
<td>512</td>
<td>62</td>
</tr>
<tr>
<td>Fibre density (kg/m$^3$)</td>
<td>18</td>
<td>18</td>
<td>44</td>
<td></td>
</tr>
<tr>
<td>Fibre strength $\sigma_f$ (MPa)</td>
<td>800</td>
<td>800</td>
<td>800</td>
<td></td>
</tr>
<tr>
<td>Fibre material density $\rho_s$ (kg/m$^3$)</td>
<td>7800</td>
<td>7800</td>
<td>7800</td>
<td></td>
</tr>
<tr>
<td>Mesh area $A_m$ (mm$^2$/m)</td>
<td></td>
<td></td>
<td>156</td>
<td></td>
</tr>
<tr>
<td>Mesh strength $\sigma_m$ (MPa)</td>
<td></td>
<td></td>
<td>450</td>
<td></td>
</tr>
</tbody>
</table>

The shotcrete demands were estimated from the max displacement, for deadweight loading and dynamic loading (Table 7-6).

The South Deep near displacement demands are high, while the Mponeng and Impala displacement demands are low. The tensile stresses determined from elongation of the shotcrete (far displacement) are low.
The deadweight demands are relatively low since the tendons are closely spaced. The demand increases with the cube of the tendon spacing.

The dynamic demands at the underground sites are calculated from the maximum measured PPV values and the tendon spacing. The demand increases with the square of the velocity and the cube of the tendon spacing. These demand values are not very high. For illustration, the typical ejection velocity design value of 3 m/s is used to determine the design dynamic loading demand. For Impala a lower values of 1 m/s is used. The values are considerably higher than the actual values.

**Table 7-6: Shotcrete demand at underground monitoring sites**

<table>
<thead>
<tr>
<th>Demand</th>
<th>South Deep Site 2 Early</th>
<th>South Deep Site 2 Late</th>
<th>Mponeng 116 level</th>
<th>Impala</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crack mechanism</td>
<td>Rectangular</td>
<td>Rectangular</td>
<td>Rectangular</td>
<td>Rectangular</td>
</tr>
<tr>
<td>Denominator</td>
<td>48</td>
<td>48</td>
<td>48</td>
<td>48</td>
</tr>
<tr>
<td>Max deflection (quasi-static)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile stress (from δ_t) (MPa)</td>
<td>0.027</td>
<td>1.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Measured Deflection δ_d (mm)</td>
<td>3</td>
<td>22</td>
<td>1.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Deadweight (roof prism)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>w (kPa)</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>2.4</td>
</tr>
<tr>
<td>m_d from deadweight (kNm/m)</td>
<td>0.16</td>
<td>0.16</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>Dynamic loading</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max measured ppv (mm/s)</td>
<td>200</td>
<td>1000</td>
<td>552</td>
<td>0</td>
</tr>
<tr>
<td>Kinetic Energy (kJ)</td>
<td>0.053</td>
<td>1.32</td>
<td>0.35</td>
<td>0</td>
</tr>
<tr>
<td>Design ejection velocity (mm/s)</td>
<td>3000</td>
<td>3000</td>
<td>3000</td>
<td>1000</td>
</tr>
<tr>
<td>Kinetic Energy Design (kJ)</td>
<td>11.8</td>
<td>11.8</td>
<td>10.3</td>
<td>0.4</td>
</tr>
</tbody>
</table>

The tensile capacity of the shotcrete, determined from beam tests is high relative to the tensile loading demand. Displacement capacities were calculated from the support spacing and displacements at peak load from the underground EFNARC test results.

The actual shotcrete capacities (m_c) were calculated for each site, based on the shotcrete and reinforcement characteristics and are listed in Table 7-7. These were calculated for mesh reinforcement, fibre reinforcement and using EFNARC panel test results.

The EFNARC moment capacity increases with the square of the thickness and is significantly higher at Mponeng (100 mm thick shotcrete) than at the other sites.
The moment capacities calculated using the fibre reinforcement values are lower than those determined using test results. This would suggest that the tensile strength of the cured cement and aggregate plays a significant role.

The potential mesh moment capacity is also estimated for the Mponeng site. It is lower than the fibre reinforcement capacity, but would improve the moment capacity, even with this relatively light mesh.

The EFNARC EA capacity increases with the square of the thickness and the 100 mm thick Mponeng site shotcrete has the highest capacity. The Impala site had no reinforcement in the shotcrete and the EA is therefore very low.

Table 7-7: Capacities of shotcrete at underground monitoring sites

<table>
<thead>
<tr>
<th>Capacity</th>
<th>South Deep Site 2 Early</th>
<th>South Deep Site 2 Late</th>
<th>Mponeng 116 level</th>
<th>Impala</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile loading</td>
<td>8</td>
<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>EFNARC $\delta_c$</td>
<td>7.4</td>
<td>7.4</td>
<td>9.3</td>
<td>1.7</td>
</tr>
<tr>
<td>EFNARC $m_c$ (kNm/m)</td>
<td>1.2</td>
<td>1.2</td>
<td>9.2</td>
<td>1.0</td>
</tr>
<tr>
<td>Fibre $m_c$ (kNm/m)</td>
<td>0.57</td>
<td>0.57</td>
<td>5.6</td>
<td></td>
</tr>
<tr>
<td>Mesh $m_c$ (kNm/m) based on $\sigma_m$</td>
<td></td>
<td></td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td>EFNARC EA Capacity (kJ)</td>
<td>2.1</td>
<td>2.1</td>
<td>10.2</td>
<td>0.22</td>
</tr>
</tbody>
</table>

Factors of safety are calculated using the calculated demands and displacement, moment and dynamic capacities determined from the EFNARC tests and are represented in Table 7-8.

At the South Deep site 2, the factor of safety for primary cracks is very high. However, primary cracks were already evident at the early stage. This indicates that pure tensile failure is unlikely and makes a relatively small contribution to the formation of primary cracks. The quasi-static $\delta_n$ factor of safety at this time is fairly high. Flexural failure may have started to occur since primary cracks were evident. It is possible that the some displacement had already occurred prior to monitoring. Steel fibre reinforced shotcrete has considerable residual strength and can continue to deform after the peak capacity is exceeded and can continue to contain the broken rock and prevent unravelling. At the later stage, secondary failure had occurred and there was a significant increase in the rate of displacement. This point represents a significant loss in residual strength and ultimate failure of the shotcrete.
At Mponeng only minor cracks were observed and the safety factors are high.

At Impala mine, the unreinforced shotcrete showed no signs of damage after over 700 days of monitoring. The displacement was limited to 0.5 mm, which creates a low moment demand for the shotcrete. The capacity of the unreinforced shotcrete is 2.7 times greater than the demand. Since the demand increases with the square of the velocity, the factors of safety are less than 1.0 the design demand, based on 3 m/s ejection velocities.

**Table 7-8: Factor of safety for the design sites**

<table>
<thead>
<tr>
<th>Loading</th>
<th>South Deep Site 2 Early</th>
<th>South Deep Site 2 Late</th>
<th>Mponeng 116 level</th>
<th>Impala</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quasi-static primary fracturing (δf)</td>
<td>300</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Quasi-static (δn)</td>
<td>2.5</td>
<td>0.3</td>
<td>6.2</td>
<td>3.3</td>
</tr>
<tr>
<td>Deadweight</td>
<td>2.4</td>
<td>2.4</td>
<td>24.0</td>
<td>4.6</td>
</tr>
<tr>
<td>Dynamic (Max measured ppv)</td>
<td>39.8</td>
<td>1.6</td>
<td>29.1</td>
<td>N/A</td>
</tr>
<tr>
<td>Dynamic (Design)</td>
<td>0.18</td>
<td>0.18</td>
<td>0.99</td>
<td>0.55</td>
</tr>
</tbody>
</table>

7.6 Conclusions

The following conclusions can be drawn from this analysis:

7.6.1 Yield line analysis

- The yield line analysis method is suitable for analyzing the flexural failure of shotcrete
- The envelope crack pattern is the most likely crack pattern and the moment demand should be determined using this equation.
- The moment demand for the crack patterns associated with rectangular and offset bolting are similar, however, the offset bolt pattern is marginally more favourable.
7.6.2 Shotcrete moment demand

- The modelling results in (section 5.7) can be used as a preliminary estimate of the quasi-static displacement, where measurements are not available. This indicates the amount of deformation that the shotcrete is expected to undergo. This may exceed the peak capacity and the shotcrete will then have a lower residual capacity.

- The rock load demand from deadweight loading is relatively low for the tendon spacing (1.0 m to 1.5 m) used in the underground sites. However, the rock load demand increases with the cube of the spacing and this will become a concern for greater tendon spacing.

- The dynamic loading demand can be estimated by assessing the kinematic and potential energy of a prism of rock formed between tendons. The demand increases with the cube of the spacing and the square of the ejection velocity.

7.6.3 Shotcrete capacity

- The deflections at peak load ($\delta_p$) in both EFNARC and ASTM C1550/RDP tests are highly variable, but can be used as an indication of whether the peak load is likely to be exceeded.

- The peak moment capacity of steel fibre reinforced shotcrete can be determined reliably using ASTM C1550/RDP test results.

- The peak loads and moduli in EFNARC steel fibre reinforced tests are quite variable and this is probably due to the non-unique crack pattern. The moment capacities are however, similar to the ASTM C1550/RDP test results.

- Unreinforced shotcrete and polypropylene fibre reinforced shotcrete have highly variable peak moment capacities.

- Peak moment capacity is proportional to steel fibre density or mesh area and increases with the cube of the shotcrete thickness.

- The residual moment capacities can be reliably determined for both steel and polypropylene fibre reinforced shotcrete. The residual strength increases with increasing fibre content.
- Steel fibre reinforced shotcrete will control the displacement rate more effectively than polypropylene fibre reinforced shotcrete. Polypropylene fibre reinforced shotcrete can accommodate larger deformations.

- Unreinforced shotcrete does not have residual strength and therefore should not be used where quasi-static deformation or dynamic loading is expected.

- Energy absorption increases with increasing fibre content.

- Steel fibre reinforced shotcrete has higher Energy absorption capacity prior to any deformation taking place, but after some initial deformation has taken place, the remaining energy absorption capacity of polypropylene fibre reinforced shotcrete is higher.

- The potential corrosion of steel fibre after cracking should also be considered.

- The shotcrete capacity increases with increasing thickness.

- Actual underground capacities are highly variable and dependant on the local rock surface geometry and shotcrete application.
8 Proposed Shotcrete Design Methodology for application in underground operations

The objective of this chapter is to describe a practical shotcrete design methodology that can be applied in underground mines. It is designed to be accessible to rock engineering practitioners on operations and makes use of simple design charts. The important practical considerations in the application of this methodology are discussed. Detailed justifications are not provided, since these have been discussed in detail in the preceding chapters.

The first step in the process is to define the rock mass and loading conditions as illustrated in Figure 8-1 and determine the expected ground response. This is discussed in section 8.1. Then the designer should follow the design flow chart in Figure 8-2.

- The first question asked is whether areal coverage is required or not (section 8.2.)
- Then the potential Quasi-static loading and deformation should be assessed (section 8.3).
- The potential for adhesion failure should be checked (section 8.4)
  - If there is no adhesion loss, design for direct shear failure under deadweight loading (section 8.5)
  - If there is adhesion loss then design for flexural failure under deadweight loading (section 8.6)
- If dynamic loading is expected then design for dynamic loading (section 8.7)

Each of the loading mechanisms is discussed in terms of demand and capacity. A factor of safety can be determined as follows:

\[
\text{Factor of safety} = \frac{\text{Capacity}}{\text{Demand}}
\]

Recommended factors of safety are provided for each loading condition.

The design chart includes check boxes, which question the practical aspects and costs of any improvements to the shotcrete design. Where it is not practical or cost effective to improve the shotcrete design, alternative or additional means of support should be considered.

It is important that monitoring of implementation of the design is carried out and this is discussed in section 8.10.
<table>
<thead>
<tr>
<th>Rock Mass Condition (GSI)</th>
<th>Loading Condition</th>
<th>Low Stress</th>
<th>Moderate stress</th>
<th>High Stress</th>
<th>Dynamic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive (&gt;70)</td>
<td>A1</td>
<td>Limited or no damage.</td>
<td>Fracturing and minor spalling of rock occurs.</td>
<td>Intense fracturing, spalling and bulking of rock occurs.</td>
<td>Violent ejection of rock.</td>
</tr>
<tr>
<td>Jointed (40-70)</td>
<td>B1</td>
<td>Blocks or wedges mobilize under gravity loading.</td>
<td>Fracturing is altered by joint pattern. Blocks or wedges begin to slide.</td>
<td>Intense fracturing and sliding along joints. Blocks or wedges slide.</td>
<td>Violent ejection of rock.</td>
</tr>
<tr>
<td>Heavily jointed (&lt;40)</td>
<td>C1</td>
<td>Unravelling of small blocks.</td>
<td>Unravelling of small blocks. Failure can propagate into the rock mass if not controlled</td>
<td>Unravelling and crushing of blocks. Sliding along joints can create squeezing conditions</td>
<td>Violent ejection of rock and unravelling</td>
</tr>
</tbody>
</table>

**Figure 8-1:** Expected ground response for different loading conditions and rock mass characteristics (modified from Hoek, Kaiser and Bawden, 1995)
Figure 8-2: Flow chart for shotcrete design in underground mines
8.1 Inputs required for the design of shotcrete

The first step in the process is to determine the inputs required for the design of shotcrete, which are:

- Field stress and stress changes
- Seismicity
- Rock mass characteristics
- Excavation characteristics and requirements
- Tendon support design
- Shotcrete characteristics

8.1.1 Field stress

Elastic modeling should be carried out to determine the field stress. The following inputs are required for elastic modeling.

- Virgin stress state
- Elastic parameters
- Mine geometry
- Mining sequence and layout

The maximum tangential stress should be determined for the excavation as follows:

\[ \sigma_{\text{max}} = 3\sigma_1 - \sigma_3 \]

where:

\( \sigma_1 \) and \( \sigma_3 \) are the major and minor principal stresses.

It is important to use the subsidiary principal stresses in the plane of the excavation cross-section.

The stress changes during the life of the excavation must be evaluated. These are used to determine the maximum expected displacements under quasi-static loading.
8.1.2 Seismicity

The seismic environment should be determined from back analysis of seismic activity in the vicinity of the excavation of interest. Specific elastic modelling can be carried out to determine the maximum expected magnitude for a given geological structure. The objective of these analyses is to determine a design seismic ejection velocity or peak particle velocity (PPV). PPV can be determined from the distance \( R \) between the hypocentre (actual or from modelling) and the excavation as follows:

\[
PPV = 0.758(M_L) - 1.528(\log R) + 3.375 \quad \text{[SSI]}
\]

The ground vibration at the surface of the excavation is typically 2 to 10 times the PPV.

8.1.3 Rock mass characteristics

The rock mass characteristics should be determined for the geotechnical domain or ground control district in which the tunnel is situated. Important aspects to consider are:

- Type of rocks
- Rock strength
- Geological structures
- Bedding and joint characteristics
- Weathering
- Ground water

Rock mass classification should be carried out to determine the Geological Strength Index (GSI). This is used directly in the determination of the maximum expected displacement under quasi-static loading.

8.1.4 Excavation characteristics and requirements

It is important to consider not only the excavations, but the function and importance of an excavation. The following factors should be considered.

- Actual height and width
- Life of excavation
- Exposure of personnel
- Access and importance
- Minimal functional dimensions for tramming
- Accessibility for maintenance and rehabilitation

### 8.1.5 Tendon support design

The spacing between tendon supports is required for design of shotcrete. This is used to determine the demand under quasi-static loading, deadweight and dynamic loading. Figure 8-3 shows the expected crack pattern for a rectangular tendon pattern (section 7.2). The rectangular pattern is used for determining demand, since this has the least moment resistance, is likely to occur and has a simple closed form solution. The parameters a and b represent the larger and smaller tendon spacings respectively.

![Tendon spacing and rectangular crack pattern](image)

**Figure 8-3:** Tendon spacing and rectangular crack pattern

### 8.1.6 Shotcrete characteristics

The length, shape and material properties of fibres have a significant effect on capacity of shotcrete (Kirsten 1992, Kirsten, 1998 and Keyter and Kersten, 2001) and it is therefore critical
that a panel test programme is carried out as part of the shotcrete design process. The ASTM C1550 round panel test is recommended, since it has a determinate crack pattern, provides more consistent results and the ultimate deflection is 40 mm as opposed to 25 mm.

The following parameters should be determined from the panel tests (see section 7):

- Deflection at peak load
- Peak load
- Load deflection curves
- Remaining energy absorption curves

The test panels should be properly prepared according to the required specification. It is recommended that at least 5 tests should be carried out on each mix to determine the variability. These tests should be conducted even if the shotcrete mix has no reinforcement. The lower bound 20 percentile deflection and peak load should be used for design purposes.

It is imperative that shotcrete panels are prepared by spraying and not casting. The fibre arrangement and mix characteristics will differ if the shotcrete is not prepared in the manner it will be sprayed underground.

Mesh reinforced shotcrete can also be prepared in sprayed shotcrete panels.

In combination with the panel tests, it is essential to carry out the following tests for quality control purposes:

- Uniaxial Compressive Strength (UCS)
- Fibre content

The UCS test results and fibre content must be determined to obtain representative measures of shotcrete quality for the given batch.

The test results provided in this chapter, were determined using 30 mm long, 1100 MPa, hook ended, Metalloy steel fibres and 50 mm long 275 MPa, Homo polymer, Meyco polypropylene fibres. These should only be used for a preliminary design prior to panel testing using the proposed shotcrete mixes.
8.2 Determining the requirement for areal coverage

Areal support is required when the rock mass is blocky or fractured. If the blocks or fractured fragments are smaller than the tendon spacing, these will need to be secured with some form of areal support. The requirement is therefore joint controlled or stress controlled.

8.2.1 Joint controlled

When the rock mass is blocky the following methods can be used to determine whether areal support is required:

- Underground observations of block sizes and performance of support
- Statistical keyblock analysis (JBlock)
- Empirical design charts (Figure 8-4)

The empirical chart design chart for static loading conditions presented Figure 8-4 (Stacey and Swart, 2001) is a simplification of the Barton et. al (1974) Q chart. The RMR presented on top of the chart is equivalent to the GSI + 5.

Figure 8-4: Shotcrete design chart (Stacey and Swart, 2001)
8.2.2 Stress controlled

Fracturing occurs when the stress acting on the excavation walls exceeds the strength. The following empirical criteria are commonly used to determine whether support is required:

- Field stress $\sigma_1 / \sigma_c$ ratio
- Rockwall Condition Factor ($RCF = \frac{3\sigma_1-\sigma_2}{F\sigma_c}$)

These criteria are discussed in more detail in Budavari (1983) and Jager and Ryder (1999).

8.3 Quasi-static loading and deformation

Under quasi-static loading the required support pressures are far greater than can be practically achieved using shotcrete and bolts in underground mines. In section 7.3.1, it was proposed that the maximum expected displacement is the demand and the deflection of the shotcrete should be considered when determining the capacity.

The maximum displacement can be determined from underground monitoring. However, before any monitoring has been done it is necessary to determine an initial estimate of the displacement demand in order to carry out the design.

Numerical modelling was carried out to determine ground reaction curves for 3.5 m x 3.5 m tunnels in typical Quartzite, Shale, Lava, Pyroxenite and Norite/Anorthosite with GSI values ranging from 20 to 90 under different maximum tangential stress levels (section 5.7). Maximum displacements have been determined for changes in stress and are presented in Table 8-1.

Once the maximum expected displacement has been determined, this needs to be adjusted for the actual tunnel size as indicated in Figure 8-5. The maximum expected displacement is then multiplied by the displacement increase factor.
Table 8-1: Maximum displacements for various rock masses under different loading conditions.

<table>
<thead>
<tr>
<th></th>
<th>Change in tangential stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>GSI 150-200 MPa 200-250 MPa 250-300 MPa</td>
</tr>
<tr>
<td>Quartzite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20 48.20 Squeezing Squeezing Squeezing</td>
</tr>
<tr>
<td></td>
<td>30 5.50 5.50 5.50</td>
</tr>
<tr>
<td></td>
<td>40 4.50 43.00 51.00</td>
</tr>
<tr>
<td></td>
<td>50 8.00 25.30 35.70</td>
</tr>
<tr>
<td></td>
<td>60 7.50 6.70 7.80</td>
</tr>
<tr>
<td></td>
<td>70 4.00 5.50 5.50</td>
</tr>
<tr>
<td></td>
<td>80 4.00 3.50 4.00</td>
</tr>
<tr>
<td></td>
<td>90 2.75 1.30 4.60</td>
</tr>
<tr>
<td>Shale</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20 Squeezing Squeezing Squeezing</td>
</tr>
<tr>
<td></td>
<td>30 Squeezing Squeezing Squeezing</td>
</tr>
<tr>
<td></td>
<td>40 34.20 37.20 69.50</td>
</tr>
<tr>
<td></td>
<td>50 15.20 17.80 29.70</td>
</tr>
<tr>
<td></td>
<td>60 6.50 7.80 10.10</td>
</tr>
<tr>
<td></td>
<td>70 5.30 4.30 8.30</td>
</tr>
<tr>
<td></td>
<td>80 4.29 5.93 2.47</td>
</tr>
<tr>
<td></td>
<td>90 3.70 3.30 4.13</td>
</tr>
<tr>
<td>Lava</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20 54.10 89.00 79.00</td>
</tr>
<tr>
<td></td>
<td>30 25.60 37.30 34.20</td>
</tr>
<tr>
<td></td>
<td>40 7.46 16.50 15.00</td>
</tr>
<tr>
<td></td>
<td>50 4.07 7.80 6.90</td>
</tr>
<tr>
<td></td>
<td>60 2.45 4.20 4.20</td>
</tr>
<tr>
<td></td>
<td>70 1.05 2.45 3.65</td>
</tr>
<tr>
<td></td>
<td>80 0.40 1.36 1.84</td>
</tr>
<tr>
<td></td>
<td>90 0.35 0.49 1.21</td>
</tr>
<tr>
<td>Pyroxenite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20 32.00 20.60 22.50</td>
</tr>
<tr>
<td></td>
<td>30 21.10 18.00 18.20</td>
</tr>
<tr>
<td></td>
<td>40 12.84 12.30 12.40</td>
</tr>
<tr>
<td></td>
<td>50 9.36 4.60 7.50</td>
</tr>
<tr>
<td></td>
<td>60 8.38 3.79 2.40</td>
</tr>
<tr>
<td></td>
<td>70 7.61 3.15 3.17</td>
</tr>
<tr>
<td></td>
<td>80 6.36 3.10 3.30</td>
</tr>
<tr>
<td></td>
<td>90 4.88 2.88 3.51</td>
</tr>
<tr>
<td>Norite/Anorthosite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>20 21.49 17.70 14.00</td>
</tr>
<tr>
<td></td>
<td>30 14.31 7.90 14.50</td>
</tr>
<tr>
<td></td>
<td>40 9.12 7.50 7.50</td>
</tr>
<tr>
<td></td>
<td>50 6.60 3.60 3.40</td>
</tr>
<tr>
<td></td>
<td>60 5.00 4.10 3.50</td>
</tr>
<tr>
<td></td>
<td>70 4.82 3.10 2.50</td>
</tr>
<tr>
<td></td>
<td>80 4.00 2.50 2.40</td>
</tr>
<tr>
<td></td>
<td>90 4.60 1.90 1.60</td>
</tr>
</tbody>
</table>
8.4 Adhesion failure check

Barrett and McCreath (1995) used the adhesion model proposed by Fernandez-Delgado et al (1981, 1976) to analyse adhesion failure. It must be noted that adhesion failure does not imply failure of the shotcrete, but simply makes the flexural failure mechanisms kinematically possible. If there is no adhesion failure, then the shotcrete must fail in direct shear.

When there is closely spaced fracturing parallel to the shotcrete rock interface, flexural failure can occur irrespective of the adhesion strength (see section 4.4.6). Therefore, if stress fracturing is expected, the adhesion test is not necessary and the design must be based on flexure failure.

8.4.1 Adhesion Demand

The demand is simply the load imposed on the shotcrete lining:

\[ A_d = W = \frac{p g a b^2}{2 \sqrt{3}} \]
where

\[ a, b \] are the larger and smaller tendon spacings respectively,

\[ \rho \] is the density of the rock, and

\[ g \] is the gravitational acceleration (9.8 m/s\(^2\))

**Figure 8-6:** Weight of a prism of rock versus tendon spacing

### 8.4.2 Adhesion Capacity

The capacity of a shotcrete lining to resist de-bonding \((A_c)\) for a rectangular pattern is (Figure 2-29):

\[ A_c = 2(a + b)\sigma_{sa} z_a \]

where \( \sigma_{sa} \) is the adhesive strength of shotcrete (section 2.4.2), and

\( z_a \) is the adhesive bond length, defined as the distance from the perimeter of the panel (in the plane of the lining) over which the adhesive forces act (Figure 2-29). Adhesive bond lengths are 30 mm for relatively poor adhesive strengths of 0.5 MPa to 1.0 MPa.
(Hahn and Holmgren, 1979) and 50 mm for relatively good adhesive strengths of 1.0 MPa to 2.0 MPa (Fernandez-Delgado, 1981). SABS:0100-1:1992 specifies a partial safety factor of 1.40 for bond strength, although this is for steel bars in concrete. The shotcrete rock bond strength is likely to be more variable.

**Figure 8-7: Adhesion model modified from Barrett and McCreath (1995)**

The adhesion strength can be estimated from Figure 8-8.
8.5 Direct shear failure under deadweight loading

Barrett and McCreath (1995) propose that direct shear failure should be determined using the largest block that can be formed between tendons. The shear resistance is provided by the shear failure surface formed within the shotcrete along the perimeter of the block (Figure 2-30). It is possible for smaller blocks to create shear loads on the lining, but these will always be less likely to fail, since the demand is proportional to the area or volume of the block, while the capacity is proportional to the perimeter of the block.

8.5.1 Deadweight demand

The demand \( T_d \) is simply the load imposed on the shotcrete lining (Figure 2-28):

\[
T_d = W = \frac{\rho g a b^2}{2\sqrt{3}}
\]

Figure 8-6 can be used to determine the deadweight loading.
8.5.2 Shear Capacity

The capacity \( T_c \) of a shotcrete lining to resist direct shear for a rectangular pattern is (Figure 2-30):

\[
T_c = 2(a + b)h\sigma_{ss}
\]

where \( h \) is the thickness of the shotcrete, and \( \sigma_{ss} \) is the shear strength of shotcrete in direct shear. Barrett and McCreath suggest values of between 1.0 MPa (8 hour strength) and 8.0 MPa (28 day strength). Direct shear strength tests can be conducted on shotcrete. SABS 0100-1:1992 specifies a minimum design strength of \( \sigma_{ss} = 0.75\sqrt{\sigma_{sc}} \) or 4.75 MPa (lesser of), where \( \sigma_{sc} \) is the compressive strength of the shotcrete. A partial safety factor of 1.40 is specified for shear strength.

![Figure 8-9: Direct shear model modified from Barrett and McCreath (1995)](image)

The direct shear strength can be estimated from Figure 8-10 for a **1.0 m support spacing**. With **increased support spacing, the shear capacity increases linearly**. It should be noted that direct shear failure is very unlikely.
8.6 Flexural failure under deadweight loading

The deadweight loading is due to the weight of rock in between tendon support. The peak capacity in flexural loading is used. In deep mines, where the horizontal clamping stresses are high, only smaller blocks will need to be supported between tendons and relatively thin unreinforced shotcrete or thin sprayed liners could prove adequate to maintain the integrity of the rock. Blocks that slide from the walls of the excavation create a lower driving force and the same mass of rock can be supported more easily.

8.6.1 Deadweight loading demand

The weight of the largest wedge that can be formed represents the demand or required support pressure (section 7.3.2). Barrett and McCreath (1995) propose calculating the mass of a roof prism defined by the tendon spacing and joint angles of 30° to the tendon. The moment demand \( m_d \) can be determined as follows:
where

\[ m_d = \frac{\rho g a^2 b (3b - a)}{96\sqrt{3}(a + b)} \]

\[ a, b \] are the larger and smaller tendon spacings respectively,

\[ \rho \] is the density of the rock, and

\[ g \] is the gravitational acceleration (9.8 m/s²)

Assuming a square pattern:

\[ m_d = \frac{\rho g a^3}{96\sqrt{3}} \]

This equation is presented in Figure 8-11. Different rock densities are shown. The moment demand increases linearly with the density, but with the cube of the tendon spacing (square pattern). Where no tendons are used, the excavation dimensions can be used to determine the moment demand.

\[ \text{Figure 8-11: Estimating the maximum deadweight demand} \]
8.6.2 Moment capacity of shotcrete

If the shotcrete is expected to undergo deformation (section 8.3), then it will lose capacity. In section 7.4.1 it was shown that the equivalent deflection in both EFNARC and ASTM C1550 round panels can be determined from the maximum displacement ($\delta_c$) as follows:

$$\delta_p = 0.75 \delta_c / b$$

Where

$b$ is the minimum support spacing

Figure 8-12 shows the load deflection graphs determined from ASTM C1550 round panel tests (section 6) for a range of fibre reinforcement. A similar graph should be determined using a range of shotcrete mixtures with varying fibre content (see section 8.1.6). The remaining load capacity ($W_{pc}$) can be estimated from this graph.

![Load deflection graphs for ASTM C1550 round panel tests](image)

**Figure 8-12: Load deflection graphs for ASTM C1550 round panel tests**

The residual capacity can be improved by increasing the fibre/mesh content or increasing the thickness of shotcrete applied underground.
When $\delta_p$ is small, then the peak load value should be used. However, this is difficult to determine from Figure 8-12 and it is proposed that the 20 percentile values of peak load and the deflection at peak load are assessed as described in section 7.5.1. Table 8-2 provides the peak load and deflection at peak load for ASTM C1550 round panel tests (see Table 7-2).

**Table 8-2: Peak load and critical deflection for ASTM C1550 round panel tests**

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Peak load (kN)</th>
<th>Deflection at peak load (mm)</th>
<th>Residual strength (when deflection is greater than the deflection at peak load)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>25</td>
<td>2.5</td>
<td>No residual strength</td>
</tr>
<tr>
<td>Polypropylene fibre</td>
<td>25</td>
<td>2.5</td>
<td>Read residual strength from graph ( $&gt; 5$ mm deflection)</td>
</tr>
<tr>
<td>Steel Fibre (40kg/m3)</td>
<td>25</td>
<td>3.5</td>
<td>Read residual strength from graph</td>
</tr>
<tr>
<td>Steel Fibre (55kg/m3)</td>
<td>28</td>
<td>3.9</td>
<td>Read residual strength from graph</td>
</tr>
<tr>
<td>Steel Fibre (70kg/m3)</td>
<td>40</td>
<td>6.1</td>
<td>Read residual strength from graph</td>
</tr>
</tbody>
</table>

The deadweight capacity is the moment capacity of shotcrete on the wall. This can be determined as follows:

**ASTM C1550:**

$$m_c = \frac{h^2 W_{pc}}{0.0312}$$

where:

$W_{pc}$ is the peak load (kN)

$h$ is the thickness of the applied shotcrete.

**EFNARC:**

$$m_c = \frac{h^2 W_{pe}}{0.045}$$

where:

$W_{pe}$ is the peak load (kN)

Figure 8-13 and Figure 8-14 were derived using the above equations. The peak loads can be improved by increasing the steel fibre content and the strength of the shotcrete mix.
Polypropylene fibre does not have much influence on the peak load. Increasing the thickness has a major effect on the peak moment capacity.

**Figure 8-13:** Moment Capacity from ASTM C1550 Round panels
The application of dynamic loading in shotcrete design remains quite theoretical. However, the method does provide an indication of the limits of the application of shotcrete under dynamic loading.

It considers the ejection of a block in between tendons and the ability of a shotcrete lining to absorb the energy. It is recommended that a factor of safety of at least 2.0 should be considered due to the uncertainty and the exponential increase in the dynamic loading demand.

### 8.7.1 Dynamic loading demand

The roof prism analogy used for deadweight loading (section 8.6.1) can be applied to dynamic loading (section 7.3.3). The energy of an equivalent ejected prism is taken to be the dynamic loading demand. In the case of a roof prism, both kinetic and potential energy should be considered, while for a sidewall prism only the kinetic energy should be considered.
Roof prism dynamic loading demand:

\[ E = \frac{\rho a^2 b v^2}{4\sqrt{3}} + \frac{\rho g a^2 b \delta_e}{2\sqrt{3}} \]

Sidewall prism dynamic loading demand:

\[ E = \frac{\rho a^2 b v^2}{4\sqrt{3}} \]

where:

- \( a \) and \( b \) are the smaller and larger tendon spacings respectively
- \( \rho \) is the density of the rock
- \( g \) is the gravitational acceleration (9.8 m/s\(^2\))
- \( v \) is the ejection velocity
- \( \delta_e \) is the displacement undergone before being arrested by the shotcrete:
  - ASTM C1550 round panels
    \[ \delta_e = 0.053 \ b - \delta_d \]
  - EFNARC square panels
    \[ \delta_e = 0.033 \ b - \delta_d \]
- \( \delta_d \) is the quasi-static maximum displacement

Figure 8-15 and Figure 8-16 represent the kinetic energy and potential energy dynamic loading charts. It should be noted that both the kinetic and potential energy increase with the cube of the support spacing (square pattern). The kinetic energy also increases with the square of the ejection velocity.
Figure 8-15: Dynamic loading demand (Kinetic Energy) chart
8.7.2 Dynamic loading capacity

The dynamic loading capacity can be determined from test panel energy absorption capacity (section 7.4.2 and 7.5.1). The remaining Energy Absorption capacity is the area under the load deflection curve for panel tests beyond the maximum deflection (see section 8.6.2) determined from the quasi-static displacement (section 8.3).

Figure 8-17 was determined from ASTM C1550 round panel tests (section 6). A similar graph should be determined using a range of shotcrete mixtures with varying fibre content (see section 8.1.6). This graph can be used to determine the remaining energy absorption.
A rectangular tendon pattern is assumed to determine the applied shotcrete dynamic loading capacity as follows:

ASTM C1550 Round panel:

\[ EA = 8.7E_A p_c \left( \frac{h}{0.075} \right)^2 \frac{b}{0.75} \]

where

- \( EA_{pc} \) is the ASTM C1550 round panel energy absorption
- \( h \) is the thickness of the applied shotcrete
- \( b \) is the lesser tendon spacing
EFNARC

\[ EA = 6EA_{pe} \left( \frac{h}{0.075} \right)^2 \frac{b}{0.75} \]

where

\[ EA_{pe} \] is the EFNARC panel energy absorption

The charts in Figure 8-18 and Figure 8-19 are derived using a tendon spacing of 1.0 m. If the tendon spacing is greater, the energy absorption capacity can be determined by simply multiplying by the tendon spacing. Note that while the energy absorption capacity increases linearly with increasing tendon spacing, the demand increases with the cube of the tendon spacing. The factor of safety decreases with the square of the support spacing.

Improving the reinforcement will improve the EA capacity, but panel tests must be done to determine the improved EA. The EA capacity increases with the square of the shotcrete thickness.

**Figure 8-18: Energy Absorption capacity from ASTM C1550 round panels**
8.8 Practical considerations

The steel fibre reinforcement provides very high peak and early residual loads, but these reduce quite rapidly. This is due to the high stiffness of the steel fibres. Polypropylene fibre reinforcement has very little influence on the peak load and the residual loads are initially lower, but they are maintained at the final displacement. This is due to the low stiffness of polypropylene. The energy absorption of steel fibre reinforced shotcrete is higher initially, but polypropylene shotcrete has a higher remaining energy absorption capacity after some deformation has taken place.

Steel fibre will tend to corrode if cracks form in the shotcrete, which should be considered if significant deformation is expected.

Unfortunately rebounded fibres tend to accumulate in drains and ultimately report to the pumps. Polypropylene fibres tend to cluster and block the pumps.
The higher fibre densities of 70 kg/m$^3$ (steel fibre) and 8 kg/m$^3$ (polypropylene fibre) are not commonly used and some training of nozzle operators will be required if these densities are selected for design.

It is also not common to use fibres with a length greater than 30 mm (particularly steel fibres), since they tend to block the shotcrete machine more frequently.

In deep South African mines, the transport of large volumes of shotcrete mix is a major problem. It is usually not practical to specify very thick shotcrete in order to achieve the design requirements.

### 8.9 Example applications of shotcrete design methodology

Four examples are discussed:

- Shale tunnel undergoing large stress changes
- Large excavation in shale undergoing large stress changes
- Tunnel in anorthosite undergoing moderate stress changes
- Tunnel in a blocky rock mass and no stress damage or deformation expected

#### 8.9.1 Example 1: Shale tunnel undergoing large stress changes

A 3.5 m x 3.5 m tunnel is located in footwall shale having a GSI value of 60. The density of the shale is 2500 kg/m$^3$ and the tunnel is supported by bolts on a 1.5 m x 1.5 m square pattern. During the life of the tunnel it will be subjected to a tangential stress change of 200 MPa to 250 MPa. Design appropriate shotcrete support under static and dynamic loading conditions.

**Initial checks**

1. Stress fracturing is expected and containment support will be required (section 8.2).
2. Maximum quasi-static displacement ($\delta_d$) (section 8.3, Table 8-1): 7.8 mm – no correction for size.
3. No adhesion, due to fracturing – Design for flexural failure (section 8.4)

**Design for flexural failure under deadweight loading (section 8.6)**

1. Moment Demand ($m_d$) = 6 kNm/m (Figure 8-11)
2. Equivalent panel deflection ($\delta_p$) = 3.9 mm (section 8.6.2)
3. Test for
   A. 55 kg/m³ steel fibre
   B. 6 kg/m³ polypropylene fibre
4. Remaining load capacity ($W_{pc}$) (Table 8-2 and Figure 8-12):
   A. 28 kN
   B. 10 kN
5. Required shotcrete thickness for deadweight loading (Figure 8-13):
   A. 75 mm
   B. 100 mm

**Design for dynamic loading (section 8.7)**

1. Displacement for demand ($\delta_e$) = 72 mm (section 8.7.1)
2. Potential energy demand = 2 kJ (Figure 8-16)
3. Design kinetic energy for different ejection velocities:
   A. 3 m/s
   B. 2 m/s
   C. 1 m/s
4. Kinetic energy demand (Figure 8-15):
   A. 15 kJ
   B. 6 kJ
   C. 1.5 kJ
5. Total energy demand ($EA_d$):
   A. 17 kJ
   B. 8 kJ
   C. 2.5 kJ
6. Equivalent panel deflection ($\delta_p$) = 3.9 mm (section 8.6.2)
7. Remaining shotcrete panel energy absorption (Figure 8-17): Steel fibre \(70\text{Kg/m}^3 = 610\text{ J}, 55\text{ kg/m}^3 = 510\text{ J},\) Poly \(8\text{ kg/m}^3 = 420\text{ J}, 6\text{ kg/m}^3 = 320\text{ J}.\)

8. Required shotcrete reinforcement and thickness (Figure 8-18, applying a factor of safety of 1.5):
   - A. Steel \(55\text{ kg/m}^3, 150\text{ mm thick or Polypropylene }6\text{ kg/m}^3\) 200 mm thick
   - B. Steel \(55\text{ kg/m}^3, 100\text{ mm thick or Polypropylene }6\text{ kg/m}^3\) 150 mm thick
   - C. Steel \(55\text{ kg/m}^3, 50\text{ mm thick or Polypropylene }6\text{ kg/m}^3\) 75 mm thick

**Comments**

It should be noted that the fibre densities selected are relatively high in comparison with what is in typical use underground. The required thickness of the shotcrete is greater for polypropylene reinforced shotcrete than steel fibre reinforced shotcrete. However in a corrosive environment, the steel fibre is likely to corrode after the deformation has taken place and cracks have formed, limiting the useful working life of the shotcrete. The thickness of the shotcrete required for deadweight loading is higher than that which is typically used. If the bolt spacing is reduced to 1.0m, then the shotcrete thickness will reduce to 50 mm and 75 mm for the steel and polypropylene fibre reinforcement selected.

The thickness of the shotcrete required for a 3.0 m/s ejection velocity is much greater than typically used underground. Given the difficulty in transporting shotcrete in deep mines, it is suggested that an alternate containment support is used.

**8.9.2 Example 2: Large excavation in shale undergoing large stress changes**

An 8 m wide excavation is located in footwall shale having a GSI value of 50. The density of the shale is 2500kg/m\(^3\) and the tunnel is supported by bolts on a 1.5 m x 1.5 m square pattern. During the life of the tunnel it will be subjected to a tangential stress change of 200 MPa to 250 MPa. Design appropriate shotcrete support under static and dynamic loading conditions.

**Initial checks**

1. Stress fracturing is expected and containment support will be required (section 8.2).
2. Maximum quasi-static displacement ($\delta_d$) (section 8.3, Table 8-1): 16.5 mm x 1.8 = 30 mm (corrected for size, Figure 8-5)

3. No adhesion, due to fracturing – Design for flexural failure (section 8.4)

**Design for flexural failure under deadweight loading (section 8.6)**

1. Moment Demand ($m_d$) = 6 kNm/m (Figure 8-11)

2. Equivalent panel deflection ($\delta_p$) = 15 mm (section 8.6.2)

3. Test for
   - A. 55 kg/m$^3$ steel fibre
   - B. 6 kg/m$^3$ polypropylene fibre

4. Remaining load capacity ($W_{pc}$) (Figure 8-12):
   - A. 21 kN
   - B. 10 kN

5. Required shotcrete thickness for deadweight loading (Figure 8-13):
   - A. 75 mm
   - B. 100 mm

**Design for dynamic loading (section 8.7)**

1. Displacement for demand ($\delta_e$) = 65 mm (section 8.7.1)

2. Potential energy demand = 2 kJ (Figure 8-16)

3. Design kinetic energy for different ejection velocities:
   - A. 3 m/s
   - B. 2 m/s
   - C. 1 m/s

4. Kinetic energy demand (Figure 8-15):
   - A. 15 kJ
   - B. 6 kJ
   - C. 1.5 kJ
5. Total energy demand ($EA_d$):
   A. 17 kJ
   B. 8 kJ
   C. 3.5 kJ

6. Equivalent panel deflection ($\delta_p$) = 15 mm (section 8.6.2)

7. Remaining shotcrete panel energy absorption (Figure 8-17): Steel fibre 70Kg/m$^3$ = 260 J, 55 kg/m$^3$ = 260 J, Poly 8 kg/m$^3$ = 280 J, 6 kg/m$^3$ = 240 J.

8. Required shotcrete reinforcement and thickness (Figure 8-18, applying a factor of safety of 1.5):
   A. Steel 55 kg/m$^3$, 200 mm thick or Polypropylene 6 kg/m$^3$, 200 mm thick
   B. Steel 55 kg/m$^3$, 150 mm thick or Polypropylene 6 kg/m$^3$, 150 mm thick
   C. Steel 55 kg/m$^3$, 100 mm thick or Polypropylene 6 kg/m$^3$, 100 mm thick

Comments

The comments in example 1 also apply here. It should be noted that the remaining moment and energy absorption capacity of the steel fibre reinforced shotcrete is greatly diminished by the increased deformation, while the polypropylene fibre reinforced shotcrete is much less affected.

8.9.3 Example 3: Tunnel in anorthosite undergoing moderate stress changes

A 3.5 m x 3.5 m tunnel is located in footwall anorthosite having a GSI value of 70. The density of the anorthosite is 3000kg/m$^3$ and the tunnel is supported by bolts on a 1.0 m x 1.0 m square pattern. During the life of the tunnel it will be subjected to a tangential stress change of 120 MPa to 160 MPa. Dynamic loading is not anticipated. Note that this example is similar to the Impala underground site (see Chapter 4 and section 7.5.2). Design appropriate shotcrete support.

Initial checks

1. Stress fracturing is expected and containment support will be required (section 8.2).
2. Maximum quasi-static displacement ($\delta_d$) (section 8.3, Table 8-1): 2.5 mm – no correction for size

3. No adhesion, due to fracturing – Design for flexural failure (section 8.4)

**Design for flexural failure under deadweight loading (section 8.6)**

1. Moment Demand ($m_d$) = 2 kN/m (Figure 8-11)

2. Equivalent panel deflection ($\delta_p$) = 1.9 mm (section 8.6.2)

3. Test for unreinforced shotcrete (Table 8-2)

4. Remaining load capacity ($W_{pc}$): 25 kN (Table 8-2)

5. Required shotcrete thickness for deadweight loading: 50 mm (Figure 8-13).

**Comments**

Unreinforced shotcrete will be adequate to provide the deadweight loading requirements in this environment. It should be noted that unreinforced shotcrete has very low dynamic loading capacity and this is suitable if there are no dynamic loading requirements.

**8.9.4 Example 4: Tunnel in a blocky rock mass and no stress damage or deformation expected**

A 3.5 m x 3.5 m tunnel is located in a blocky rock mass, which requires containment support. The density of the rock is 3000 kg/m$^3$ and the tunnel is supported by bolts on a 2.0 m x 2.0 m square pattern. No stress changes or dynamic loading are anticipated. Design appropriate shotcrete support.

**Initial checks**

1. The rock mass is blocky and containment support will be required (section 8.2).

2. No quasi-static displacement expected (section 8.3).

3. The weight of the rock prism to be supported: $W = 68$ kN (section 8.4.1, Figure 8-6)

4. Adhesion capacity for low bond strength ($\sigma_{sa} = 0.5$ MPa, $z_a = 30$ mm) (section 8.4.2, Figure 8-8): $A_c = 120$ kN

5. Adhesion factor of safety = 1.76 – Design for direct shear failure (Figure 8-8).
Design for direct shear failure under deadweight loading (section 8.5)

1. Direct shear demand \( (T_d) = W = 68 \text{ kN} \) (Figure 8-6)

2. Design for low shear strength (section 8.5.2): \( \sigma_{ss} = 1 \text{ MPa} \)

3. Required thickness (Figure 8-10): 25 mm, \( T_c = 100 \times 2 = 200 \text{ kN} \), factor of safety = 2.9

Comments

25 mm thick unreinforced shotcrete will be adequate to provide the deadweight loading requirements in this environment. Note that direct shear failure is very unlikely and uncommon in practice. However, it is extremely important to check that the bond strength is adequate.

8.10 Monitoring of shotcrete performance

It is important to monitor the performance to provide feedback into the design. The following aspects should be considered when designing a monitoring programme:

- Quality control
- Visual observations, and
- Displacement monitoring

8.10.1 Quality control

It is important to monitor the consistency of the shotcrete application and assess whether it continues to meet the design criteria. Poor quality will influence the performance of the shotcrete. The following aspects should be considered in a quality control programme:

- Shotcrete thickness testing
- ASTM C1550 round or EFNARC square panels
- Smaller test panels for drilling core (UCS and fibre content)
- Drilling core from the applied shotcrete (UCS and fibre content)
- Hilti penetrometer test
- Bond strength testing
**Shotcrete thickness testing**

During spraying, the most common way to measure thickness is to use nails that are the length of the required shotcrete thickness. These are pressed into the shotcrete and an impression is left by small plate welded onto the nail. A pattern of 1.0 m x 1.0 m is commonly used. It is easy to check these afterwards and confirm the depth of spraying.

After the applied shotcrete is cured, it is important to drill through the shotcrete to determine the thickness. A hand held, battery operated drill is suitable for this testing. Holes should be drilled in between the nail marks to check that the shotcrete is evenly applied. The testing intervals depend on the importance of the excavation and the actual control of shotcrete thickness by the operator.

**EFNARC square and ASTM C1550 round panels**

Ideally, test panels (preferably ASTM C1550 round panels) should be sprayed underground during the application of shotcrete. They should be allowed to cure for at least a week in the environment in which shotcrete has been applied, before transporting. The frequency of test panel preparation will depend on the importance of the excavation. In critical excavations, one test per 30 m² to 50 m² may be appropriate, but the frequency can be reduced in less critical excavations.

**UCS and fibre content testing (panels and core drilling)**

In most South African operations, the active tunnels are quite far from shaft access and the logistics involved with transporting EFNARC, and ASTM C1550 test panels can be onerous. Test panels are large, heavy and difficult to handle manually and are often damaged during transporting. Under these circumstances, it is recommended that smaller, more robust panels that can be handled and transported more easily are sprayed on site. These should be cured underground for at least a week and then transported to surface to drill cores for testing. This is important to provide early feedback on the quality and this is the easiest way to obtain core.

At a later stage, it is recommended that cores should be drilled from the applied shotcrete. The diameter of the core should be less than the thickness of the shotcrete in order to obtain representative 1:1 length to diameter samples. This is more difficult, since hand held, battery operated core drills are not very effective in extracting core. It is usually necessary to use a
more powerful core drilling machine that needs to link to a compressed air or electric power source. The frequency of drilling core from applied shotcrete will therefore be much lower than the spraying of test panels.

The UCS provides an indication of the mix consistency and should be compared with values determined during testing at design stage. Monitoring of fibre content is critical. Fibre rebound can be as high as 50%. There is invariably some fibre loss, but if this is greater than that experienced during the design testing, the capacity of the shotcrete will be lower.

**Hilti penetrometer test**

This method, developed by Hilti, is a simple method for estimating the UCS of applied shotcrete that can be carried out more frequently than coring for UCS tests. The pullout load measured in KN/mm² is then used to read an equivalent compressive strength from a calibration curve which has been prepared from UCS testing of drilled cores.

**Bond strength testing**

Bond strength testing is more critical when shotcrete is required to contain a blocky rock mass. In fractured rock masses, there is apparent bond loss due to fracturing parallel to the wall. The bond strength is determined also from a direct pullout test on a drilled core. It is essential that the core be drilled beyond the shotcrete rock interface.

**8.10.2 Visual observations**

It is important to carry out visual observations of the formation of cracks in the applied shotcrete. Primary cracks, which form and propagate independently, have a limited effect on the residual capacity of shotcrete (section 4.4.6). However, as the cracks extend and start to interact, shotcrete enters the secondary failure stage and the residual capacity is significantly reduced. It is important to identify the transition into the secondary stage of failure. Figure 7-9 shows a typical secondary stage flexural failure. The distance between cracks (e) should be less than the support spacing before this failure mechanism is likely to occur. When the length of the cracks (d) is greater than the support spacing, the shotcrete capacity will be significantly compromised.
8.10.3 Displacement monitoring

Displacement monitoring provides an important feedback into the design for quasi-static loading. Actual displacement measurements should be compared with the modelled displacements (Table 8-1) and incorporated into future designs.

Using Multiple point borehole extensometers will help to identify both the near and far displacement (section 4.4). An anchor point should be placed at the limit of the tendon working length and then additional anchors points can be placed beyond the tendon working length. The depth of the final point depends on the depth of expected displacement. Table 8-1 can provide an indication of the depth these anchors need to be placed into the rock wall. The larger the displacement the deeper the final anchor point should be placed.

Simpler single point borehole extensometers can also be used to provide information. Ideally, the anchor should be placed at the limit of the working tendon length to measure the near displacement only as this is more critical for shotcrete design.
Closure monitoring can also be used, but measurements should ideally be taken from a fixed point at 90° to the wall of the excavation, so that the displacement can be accurately measured. The distinction between near and far displacement cannot be made using this method.

8.11 Conclusions on the shotcrete design methodology

The following conclusions can be drawn with regard to this methodology:

- This design methodology developed can be readily applied to shotcrete design for underground excavations and caters for a variety of rock mass and loading conditions.
- It specifically assesses the quasi-static, deadweight and dynamic loading conditions.
- The required inputs for the design of shotcrete are listed and discussed.
- A required testing programme for determining shotcrete characteristics is described. It is important that testing is carried out, since the shotcrete capacity will depend on the fibre characteristics and shotcrete mix.
- The calculation of demand for each loading condition is described and design charts and tables are provided for each condition.
- The shotcrete capacity is determined from laboratory testing of shotcrete panels. The ASTM C1550 round panel test is recommended for design.
- Factors of safety are recommended for each loading condition, which take the uncertainty into consideration.
- Recommendations for monitoring of shotcrete performance are provided.
9 Summary and Conclusions

The following conclusions are drawn from this work:

9.1 Underground monitoring

Five underground shotcrete test sites were identified, established and instrumented at three different South African mines.

- At the two South Deep Mine sites quasi-static pillar loading led to high deformations of 70 mm and more over the 14 months of testing. Site 1 served as a trial run during which numerous lessons were learnt and improvements were made in preparation for the development of site 2. The South Deep sites were ideal for the analysis of shotcrete failure mechanisms and the shotcrete-rock interface because of the high levels of shotcrete damage that was occurring. The sites were also exposed to strong ground motions from both seismicity and nearby bench blasts, further adding to the contributions they have made.

- Instrumentation and monitoring at the Mponeng Mine 109 site had to be downscaled due to unexpected changes in the mining strategy. The site captures the performance of fibre reinforced shotcrete at deep level mining depths where over-stopping or de-stressing conditions prevail.

- The Mponeng Mine 116 test site, originally not part of the testing programme, was established to investigate the influence of strong ground motions from seismicity on the performance of shotcrete. The site was also instrumental in observing shotcrete in its early stages of failure and helped to confirm findings from the analysis of failure mechanisms from the South Deep sites.

- The Impala Platinum Mine site captured the effectiveness of un-reinforced shotcrete in controlling spalling ground conditions at intermediate mining depths where moderate stress changes are expected.
Thorough site assessments have led to the following conclusions and improved understanding.

- At intermediate depth a 50 mm thick layer of 25 MPa un-reinforced shotcrete has been shown (by the Impala test site) to effectively control tunnel deterioration where sidewall spalling is a concern.

- In deep level mining, 70 to 120 mm of 22MPa steel fibre reinforced shotcrete has been effective in stopping severe sidewall spalling whilst effectively maintaining the integrity of the tunnel during subsequent de-stressing (Mponeng 109 level). Steel fibre reinforced shotcrete has also been noted to be more resilient to corrosion than wire mesh and lacing.

- Analysis of deformations due to large seismic events at Mponeng 116 level suggests that induced deformation (and damage to shotcrete) is related to the calculated PPV of the seismic event responsible for the strong ground motion. Damage is however only expected with PPV’s greater than a site specific threshold value.

- Instantaneous jumps in sidewall deformation measured at the South Deep sites at the time of nearby blast events show that blasting activity can be a particularly strong inducer of damage. The vibrational intensity of a blast event has been found to affect the degree of deformation that results. At South Deep damage from seismicity in the area was virtually insignificant when compared to damage from nearby bench blasting.

- In-depth analysis of shotcrete crack formation and propagation at the South Deep sites identified that shotcrete fails in two distinct stages.
  - The primary stage of failure is identified by the formation and propagation of individual “primary” cracks throughout the shotcrete installation. This stage of failure is not believed to result in a noticeable drop in the performance of installed shotcrete.
  - The secondary stage follows once primary cracks have propagated far enough to join or interact. In many cases the joining of primary cracks is achieved through the development of secondary cracking. This stage of shotcrete failure is synonymous with a marked drop in the performance of the applied shotcrete and is accompanied by significant increases in sidewall deformation and damage.

- The exact mechanism behind the formation of cracks during the primary stage has not been conclusively determined but it is known that cracks form in tension. It is further
understood that the damage caused by these cracks and its influence on the overall shotcrete performance is localised and thus rather limited.

- Cracking during the secondary stage of shotcrete failure has been observed to commonly occur according to the flexural failure mechanism. Many examples of secondary cracking according to a mechanism resembling punching-shear have also been documented. In both cases the cracks still form in tension. Secondary cracking has a substantial influence on the overall performance of shotcrete.

- Analysis of MPBX deformations taken after shotcrete failure of the secondary stage suggest that only the area falling outside of the assumed zone of influence of installed support tendons is being supported by the shotcrete.

### 9.2 Laboratory testing

Various properties of fibre reinforced shotcrete were assessed. The laboratory test results have revealed that the incorporation of fibres in shotcrete has little effect on the modulus of rupture and the uniaxial compressive strength of shotcrete. However, the incorporation of fibres improves the post peak performance of shotcrete by offering it residual strength and energy absorption capacities.

The shape of the specimens has an effect on the performance of shotcrete. This is especially the case with EFNARC panels due to the nature of the testing method, which requires the panel base to be flush with the stand on which the panel is placed during testing.

It has been noted that the performance of fibre reinforced shotcrete depends on the types of fibres used in the shotcrete.

- The post peak load behaviour of shotcrete was displayed through toughness of the ASTM beams, residual strength for the EFNARC beams and energy absorption of RDPs and EFNARC panels.

- The Round Determinate Panels (RDP) displayed less scattered load-deflection results compared to the EFNARC panels. Therefore the RDP testing method appears to produce better results in terms of repeatability of the test method and may be the best method for use in the post-peak load bearing capacity assessment of shotcrete during design. The design of shotcrete could be based on the post peak load bearing capacity, which has a direct influence on the energy absorption of shotcrete.
• The toughness index results obtained from the ASTM beam tests have shown that the increase in fibre density in shotcrete has a negligible effect on the toughness of shotcrete. This is probably because the ASTM beam testing method has a very low deflection range of up to 2mm and the range may not give a clear picture on the influence of fibres. This may require the use of a testing machine that loads the beam to complete failure. For the current ASTM test method, the toughness indices results may be used to check whether the shotcrete meets set standards, for example, the toughness category ratings by Morgan (1988) reproduced in Table 6-5 in Section 6.4.1.

• The residual strength values obtained from the polypropylene fibre reinforced EFNARC beams show no linear trend as the fibre density increases. This may also be attributed to the deflection of the beams which was up to 4mm according to the test method.

9.3 Non-linear modelling of ground reaction curves

• A generic model has been created using the discrete element code UDEC, applying a voronoi tessellation scheme to permit examination of the effect of rock fragmentation on support requirements.

• The model has been used to examine the relationship between support pressure and rock wall deformation in terms of Ground Reaction Curves (GRC) for a range of common rock mass circumstances and stress regimes that occur in both the gold and platinum/chrome tabular mines. A summary of maximum sidewall deformations is presented. These are related to rock type, rock mass rating in the form of GSI, and an estimate of the maximum stress concentration induced in the tunnel wall. The listed values are derived from models of 3.5 x 3.5 m tunnels. For other excavation sizes, a scaling factor can be applied.

• For selection of appropriate shotcrete systems for a tunnel in a certain rock mass under an estimated stress field the Ground Reaction Curves can be used to supply required support pressures to limit deformations, as well as maximum movements in the tunnel wall. Considerations and modifications to the basic GRCs include scaling for excavation size, potential for extreme squeezing (deformations are large and relatively indeterminate), and whether the tunnel will be subjected to stress change, including de-stressing.
With excavation support requirements defined in terms of deflection, estimates of required shotcrete strength, thickness and deformability can be made using yield line theory.

9.4 Yield line analysis of shotcrete

9.4.1 Yield line analysis

- The yield line analysis method is suitable for analyzing the flexural failure of shotcrete.
- The envelope crack pattern is the most likely crack pattern and the moment demand should be determined using this equation.
- The moment demand for the crack patterns associated with rectangular and offset bolting are similar, however, the offset bolt pattern is marginally more favourable.

9.4.2 Shotcrete moment demand

- The modelling results (in section 5.7) can be used as a preliminary estimate of the quasi-static displacement, where measurements are not available. This indicates the amount of deformation that the shotcrete is expected to undergo. This may exceed the peak capacity and the shotcrete will then have a lower residual capacity.
- The rock load demand from deadweight loading is relatively low for the tendon spacing (1.0 m to 1.5 m) used in the underground sites. However, the rock load demand increases with the cube of the spacing and this will become a concern for greater tendon spacing.
- The dynamic loading demand can be estimated by assessing the kinematic and potential energy of a prism of rock formed between tendons. The demand increases with the cube of the spacing and the square of the ejection velocity.
9.4.3 Shotcrete capacity

- The deflections at peak load ($\delta_p$) in both EFNARC and ASTM C1550/RDP tests are highly variable, but can be used as an indication of whether the peak load is likely to be exceeded.

- The peak moment capacity of steel fibre reinforced shotcrete can be determined reliably using ASTM C1550/RDP test results.

- The peak loads and moduli in EFNARC steel fibre reinforced tests are quite variable and this is probably due to the non-unique crack pattern. The moment capacities are, however, similar to the ASTM C1550/RDP test results.

- Unreinforced shotcrete and polypropylene fibre reinforced shotcrete have highly variable peak moment capacities.

- Peak moment capacity is proportional to steel fibre density or mesh area and increases with the cube of the shotcrete thickness.

- The residual moment capacities can be reliably determined for both steel and polypropylene fibre reinforced shotcrete. The residual strength increases with increasing fibre content.

- Steel fibre reinforced shotcrete will control the displacement rate more effectively than polypropylene fibre reinforced shotcrete. Polypropylene fibre reinforced shotcrete can accommodate larger deformations.

- Unreinforced shotcrete does not have residual strength and therefore should not be used where quasi-static deformation or dynamic loading is expected.

- Energy absorption increases with increasing fibre content.

- Steel fibre reinforced shotcrete has higher Energy absorption capacity prior to any deformation taking place, but after some initial deformation has taken place, the remaining energy absorption capacity of polypropylene fibre reinforced shotcrete is higher.

- The potential corrosion of steel fibre after cracking should also be considered.

- The shotcrete capacity increases with increasing thickness.
• Actual underground capacities are highly variable and dependant on the local rock surface geometry and shotcrete application.

9.5 Design methodology

• From the research carried out in this project, a shotcrete design methodology has been developed, which caters for a variety of rock mass and loading conditions and can be readily applied to shotcrete design for underground excavations.

• It specifically tests the quasi-static, deadweight and dynamic loading conditions.

• The required inputs for the design of shotcrete are listed and discussed.

• A testing programme for determining shotcrete characteristics is described.

• The demand for each loading condition is described and design charts and tables are provided for each condition.

• The shotcrete capacity is determined from laboratory testing of shotcrete panels. The ASTM C1550 round panel test is recommended for design.

• Factors of safety are recommended for each loading condition, which take the uncertainty into consideration.

• Recommendations for monitoring of shotcrete performance are provided.
10 Recommendations for further research

The following is recommended for further research:

- There is scope for further study into a relationship between PPV and induced instantaneous deformation. The existence of a “no deformation” threshold needs to be confirmed and the relationship after this threshold needs to be quantified.

- There is scope to further study the depth of fracturing in deep level, high deformation environments. Large variations have been noted to exist between depths of fracturing as measured using different types of instruments. It is believed that the intensity of fracturing is probably locally variable and the behaviour is more complex than may be expected.

- The concept of the primary and secondary stages of shotcrete failure deserves further testing. If possible, back analysis of underground failure incidents should be conducted, assessing the stage, and degree, of shotcrete failure at the time of site collapse.
  
  o There is scope to develop a surface test under controlled laboratory conditions that can effectively demonstrate and repeatedly produce a progression from primary to secondary stage failure as defined in this research.
  
  o If such a test can be devised then it would be useful to conduct a series of tests that investigate the effect of varying common design parameters like strength, thickness and type of reinforcement on the duration of the primary stage of failure as identified by this work. Can the shotcrete be engineered to effectively prolong the time before entering the secondary stage?

- A further programme of large scale panel tests should be carried out, with varying tendon spacing, shotcrete thickness, different types and quantities of reinforcement for uniformly distributed and point loads. The data should be analysed with yield line theory.

- A small round panel test method should be developed for quality control purposes. This will be easier to transport underground.

- A dynamic test programme should be developed with a basic test configuration that can easily be quantified. This should also include panel tests with varying tendon spacing, shotcrete thickness and different types and quantities of reinforcement.
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