MSc (Eng) RESEARCH REPORT

DESIGNING A FEASIBLE METHODOLOGY FOR SELECTING PERMANENT AREAL SUPPORT FOR VARYING ENVIRONMENTS IN UNDERGROUND MINES

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

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Declaration

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

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Abstract

Detailed evaluations of ten permanent areal support systems in different mining environments were carried out including comprehensive photographic records, of the support performance and installation. The data obtained at these sites was used to develop a methodology for selecting areal support systems in different mining environments. This methodology includes the evaluation of support performance, practicality and installed cost. Support performance combines with the support capacity, in terms of initial stiffness, peak load and yield, and performance factors (installation quality, equipment damage, blast damage and corrosion). Practical aspects of transport and installation can be assessed using the methodology and the installed support cost can be determined. The methodology provides a comprehensive, practical approach to assessing permanent areal support systems. The mining environment plays a major role in the support performance and practicality of support transportation and installation.

This project report is dedicated to my parents, Ali and Inesi, and sisters Caroline and Idah who have been a constant source of support and encouragement during the challenges of graduate school and life. I am truly thankful for having you in my life.

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  - Harmony Gold Mine (Bambanani)
  - Northam Platinum Mine (Booysendal)
  - Sibanye Gold Mine (Kloof 4 Shaft)
  - AngloGold Ashanti (Tau Tona)
  - ARM – Impala Platinum Mine (Two Rivers Platinum)
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  - Lonmin Platinum Mine (Karee 3 Shaft)
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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABBREVIATIONS, ACRONYMS, NOMENCLATURE and SI UNITS

Abbreviation	Description
A_c	Adhesion capacity
CoP	Code of Practice
D_{20}	Displacement at 20 % of design load
dbs	depth below surface
δ_d	Quasi-static displacement
D_d	Design load displacement
DMR	Department of Mineral Resources
D_o	Intersection of the line and the x-axis used to calculate the stiffness
FoS	Factor of Safety (capacity / demand)
FRS	Fibre-reinforced shotcrete
GSI	Geological Strength Index
JV	Joint Venture
K_s	Screen stiffness
kN	kiloNewton
L_{20}	20% of the design load
L_d	Design load
m_d	Moment (demand), kNm/m
MHSC	Mine Health and Safety Council
σ_a	Adhesive strength (lab test results), MPa
σ_p	Shotcrete panel deflection
σ_{sa}	Shotcrete adhesive bond strength, MPa
σ_s	Shear strength
σ_t	Tensile strength (MPa)
s	Block width (m)
SIMRAC	Safety in Mines Research and Advisory Committee
RYHP	Rapid Yielding Hydraulic Props
T_c	Direct shear capacity (tau), MPa

Abbreviation	Description
T_d	Direct shear demand (tau), MPa
t	Thickness (mm)
TSL	Thin Spray-on Liner
UG2	Upper group 2
vs	versus
W	Weight (kN)
w_b	Bond width (mm)
W_{pc}	Peak load capacity
za	Shotcrete adhesive bond length
FLAC	Fast Lagrangian Analysis of Continua
UDEC	Universal Distinct Element Code
RS2	Rock and Soil 2-dimensional
ASTM	American Society for Testing and Materials
EFNARC	European Federation of National Associations Representing for Concrete

Where not stipulated, units of measure are presented in S.I. (metric, System Internationale).

1. INTRODUCTION

Prior to excavation, a rockmass and stresses are in a state of equilibrium. However; upon excavation, the stresses are redistributed to produce a new state of equilibrium. The process of stress redistribution and exposing of the rock face by blasting or machine rock cutting may result in instabilities of excavation walls resulting in rock fall related accidents.

Rock fall related accidents are the single largest contributor of injuries and fatalities in the South African mining industry. An analysis of accidents that occurred between 2010 and 2015 shows that 31 % of the fatalities were as a result of rock falls (MHSC, 2016). Most of the rock-related mining accidents occur in the active mining faces; this is inevitable since workers spend most of the shift time in the face. Adams & Baker (2002) (cited by Jjuuko & Kalumba, 2014) state that accident statistics in the South African mines between 1998 and 2002 showed that 64 % of injuries and 67 % of fatalities caused by rock falls occurred at the stope face.

An analysis of mining accidents on a sector-by-sector basis revealed that most fatalities occur in the gold and followed by the platinum sector (MHSC, 2016). This is attributed to amongst other reasons, the increased mining depths in excess of 3 km in gold mines and rock bursts most likely occur at such depths. Moreover, most gold and platinum operations are conventional and semi-mechanised, thus highly labour intensive. As a result, there is increased risk of fatalities and injuries because of high exposure of mineworkers. Therefore, remedial action needs to be implemented; particularly in the mining faces and the improvement in rock support is one such action.

Support, a term which refers to procedures and materials used to improve stability and maintaining load bearing capacity of the rockmass near the skin of excavations. Windsor & Thompson, (1993) drew a distinction between rock reinforcement or active support, where the support elements are an *integral* part of the rock mass, and rock support or passive support, where the supporting members are *externally* applied to the rock and generate reaction when there is displacement in the rock mass. Rock support also referred to as areal support, may be installed discretely on an excavation surface, for example a grout pack and is referred to as “point areal support”. Furthermore, areal support can be installed continuously on the excavation surface, for example shotcrete and is referred to as “continuous areal support”.

According to Potvin et al (2001), over 90 % of rockfall injuries involve rocks smaller than one tonne. Another study of 273 rock-related injuries and fatalities in metal mines in USA by NIOSH in 1996, found that over two thirds involved rocks of less than 12 kg (Spearing, et al., 2009). Research during the early 1990s showed that about 80% of all rock falls in underground mines in South Africa involved less than a 1.0 m thickness of rock (Spearing, 1990; Roberts, 1991). Considering these statistics, the need for continuous areal support cannot be overemphasized. The focus in this research will be on continuous areal support and for simplicity purposes; it shall be referred to as areal support in this document.

The mining industry through the Mine Health and Safety Council (MHSC) is of the opinion that the high occurrence of rock fall accidents can be reduced significantly through the installation of long-term permanent areal support.

1.1. Definition of problem

Safe profitable mining is of paramount importance and in achieving this objective, the effective management of rock mass stability is vital. A variety of approaches to manage rock mass stability exist and these may be a function of the mining environment; mining method; exposure of personnel; geotechnical rock mass conditions and/or support strategies.

Areal support systems have been used widely in the mining industry (Potvin, 2002) and it is therefore expected that there are numerous areal support types. Whilst a pool of support units to choose from is positive for the user, challenges arise when one has to decide on the “most suitable” support unit for a particular environment.

Mine based literature survey has shown that methodologies for the design and selection of permanent areal support remain largely subjective. The selection of an appropriate support system from a range of available units requires objective assessments.

Recognising the need for a more structured approach to selecting areal support for particular mining environments, the MHSC, through the Safety in Mines Research and Advisory Committee (SIMRAC), initiated a research project to address the concern. The project aimed to design a feasible methodology for selecting permanent areal support in varying underground mining environments.

1.2. Research objectives

The primary objective of the research is to design a feasible methodology for the selection of permanent areal support in varying environments in underground mines. The methodology to meet this research objective will be achieved by doing the following:

- *A survey of literature detailing available technologies used as permanent areal support including a description of the areal coverage abilities and limitations for varying mining environments used in South African and international mines;*
- *An investigation of permanent areal support technologies used in the mines including clear visual records of the performance of such support types and documenting the successes and failures;*
- *Developing an assessment tool for evaluating the effectiveness of the technologies; and*
- *Developing an effective methodology for selection of permanent areal support for varying environments.*

1.3. Scope

The scope of the project was set out during a start-up workshop held at the commencement of the project with the aim of engaging Rock Engineers from the South African mining industry. It was argued during the workshop that off-reef areal support had received a degree of attention and that the project should focus on on-reef support, in tabular, conventional and semi-mechanised (bord and pillar), gold and platinum mines in South Africa.

This argument is also supported by the statistics quoted earlier. A summary of the inclusions and exclusions of the project are presented in Table 1-1.

Table 1-1: Project inclusions and exclusions

<table>
<tr><th colspan="3">PRACTICAL</th><th rowspan="2">THEORETICAL</th></tr>
<tr><th colspan="2">INCLUSIONS</th><th>EXCLUSIONS</th></tr>
<tr><td colspan="2">Permanent areal support</td><td>Temporary areal support</td><td rowspan="8">Literature review of the functions of areal support and the design methodologies of shotcrete, TSL and wire mesh.</td></tr>
<tr><td colspan="2">Underground</td><td>Surface</td></tr>
<tr><td colspan="2">On – reef stoping (including on-reef development)</td><td>Off-reef (development)</td></tr>
<tr><td>Conventional</td><td>Room and Pillar</td><td></td></tr>
<tr><td>Intermediate – deep level: gold</td><td>-</td><td></td></tr>
<tr><td>Shallow – intermediate: platinum</td><td>Shallow to intermediate</td><td></td></tr>
<tr><td colspan="2" rowspan="2">Underground observations, checklists</td><td>Numerical analysis</td></tr>
<tr><td>Laboratory test analysis (drop tests, material property tests, areal coverage support resistance)</td></tr>
</table>

It is important to note that the objective of the mine visits was to record the design methodology currently in place, for compiling a practical and relevant reference for selecting permanent areal support in the target environments. Without validating or checking the accuracy of the designs currently in place.

1.4. Facilities

Underground observations were done at champion mines that were identified during the project initiation workshop. The areal support systems that were observed are routinely used by the mines or have been trialled at the mining site. Research sources and project writing material are available from the University of the Witwatersrand Libraries and online resources.

1.5. Contents of research

This research report, in which the designing of a feasible methodology for selecting permanent areal support for varying environments in underground mines is the central theme, is presented in six chapters. The first chapter is introductory and details the justifications and reasons for the research described in this report.

A review of literature will be carried out for permanent areal support methods currently in use in local and international underground mines and is presented in Chapter 2. Chapter 3 will discuss the underground observations carried out and the detailed template sheet that was developed for data capturing. Chapters 4 and 5 will detail the methodology for selection of permanent areal support and ranking tool for the selection of permanent areal support in varying underground mines respectively. The outcomes and conclusions are presented in Chapter 6. A summary of underground observations at each of the mining sites is given in the appendices as well as assumptions and corrections used in the ranking tool.

2. LITERATURE REVIEW

2.1 Function of areal support

Kuijpers, (2008) states that in extreme cases single rock layers can detach from the rest of the rockmass. This kind of rock failure is supported by pinning the layer to the solid rockmass by way of rock bolts (suspending) or props. However, rock masses often contain discontinuities and/or stress fractures and these result in potential rock fragments that can dislodge in between tendons thereby requiring areal support.

The unstable rock mass has an inertial load that is transmitted to the main support units such as rock bolts. In an environment where the rock layer is not fragmented, the assumption is that 100 % of the inertial load is transmitted to the main support units (Kuijpers, 2008). Whereas in the environment where fragmentation occurs, there is limited transmission of the load to the main support. In the absence of areal support, the limited transmission capability of the inertial load can result in the rock mass instability in the form of key-block failures.

Kuijpers, (2008) explains that the function of areal support is to improve the "self – supporting" capabilities of the fragmented rock mass through transmission of a design load to the primary units via the rockmass as illustrated in Figure 2-1.

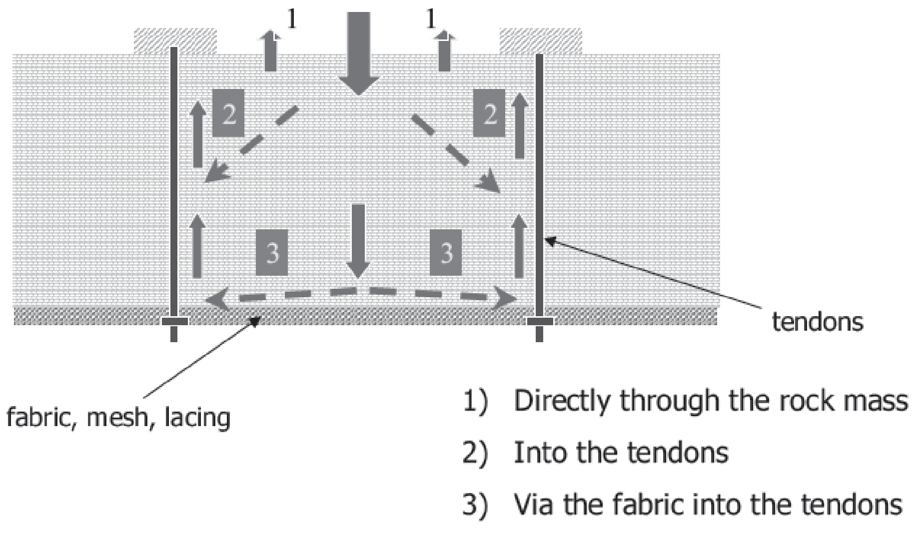

Figure 2-1: Transmission of inertial forces through a rockmass retained by tendons (Kuijpers, 2008).

The support system illustrated in Figure 2-1 resists failure of the rock mass by three mechanisms. (1) A percentage of the inertial force acting on a particular fragmented rock volume is resisted by the rock mass itself. The resisting force depends on the volume and degree of fragmentation, confining forces and the properties of the interface between the fragmented units and the solid rock mass (Kuijpers, 2008). (2) The shear resistance between the fragmented rock and tendons will aid the failure resistance and (3) finally the fragmented rock mass movement is resisted by the areal support where movement will be transferred back into the tendons.

When numerous discontinuities in a rock mass intersect, blocks of irregular shapes and sizes are created. When an excavation is made, many blocks are formed with added surfaces (Goodman & Shi, 1985). Some of these blocks will not be able to move into the free space of the excavation, either by virtue of their shape, size, or orientation, or because they are prevented from moving by others.

A few blocks called "*key blocks"* are immediately in a position to move, and as soon as they have done so, other blocks previously restrained will be liberated. Consequently, it is important to support potential "*key blocks"* or other rock fragments in between tendons that may fall and hurt the workers by way of areal support. The most commonly used permanent areal support units in South African mines are shotcrete, Thin Spray-on Liners (TSL) and wire mesh.

2.2 Shotcrete

Shotcrete is a mixture of cement, sand, small aggregate and additives (e.g. accelerators, plasticizers, etc) that is pneumatically sprayed and compacted under high velocity onto a rock surface (Hoek, 1999). There are two types of shotcrete application methods in use, namely, dry-mix shotcrete and wet-mix shotcrete. The main difference between the two systems is the stage at which water is applied to the dry ingredients, i.e. at the nozzle for dry-mix shotcrete and during mixing for wet-mix shotcrete.

Over the years shotcrete technology has been improved. Hoek, (1999) notes that the most significant improvement has been the addition of steel or polypropylene fibre reinforcement and the addition of silica fume. Silica fume, a by-product from the ferro-silicon metal industry is used as a pozzolan. Pozzolans are cementetious materials which react with calcium hydroxide produced during the hydration of cement (shotcrete). Numerous benefits of using pozzolans in shotcrete have been noted such as improved compressive strengths, minimized rebound, improved bond with rock substrate and better flexural strengths (Hoek, 1999).

Plain shotcrete is brittle and fails at low rock displacements (Jager & Ryder , 1999). To improve ductility, shotcrete has been used in conjunction with wire mesh, but this results in difficult logistical issues since large quantities of bulk materials need to be transported to the working places. Therefore, reinforced shotcrete (steel or fibre) has been used as replacement for the mesh reinforced plain shotcrete.

2.2.1 Shotcrete design methodology

Methods have been developed for the design of shotcrete. There are empirical design methods based on the Q rock mass classification system and the Norwegian Tunnelling Method (Barton, et al., 1974; Barton, 2002). The empirical shotcrete design methodology based on the Q rock mass system was later modified to suit mining applications (Stacey & Swart, 2001).

Ground reaction curves have been used as a first estimate for the determination of support demand. The corresponding support capacities for shotcrete and/or concrete lining have been subsequently presented (Hoek, 1998). This method however requires the determination of excavation deformation to construct reliable ground reaction. Furthermore Papworth, (2002) (cited by Joughin, et al., 2012) discusses the use of ground reaction curves and support interaction using support reaction curves derived from panel tests. Although the approach is sound and logical the conversion methods from panel tests to actual load deformations are not clear.

Some numerical modelling software can allow for the analysis of liners (e.g FLAC, UDEC, RS2) as beam elements attached to the rock. In relation to the rockmass, the linings are very thin and they require a high resolution, i.e. fine meshing to achieve meaningful results. Joughin, et al., (2012) conclude that the analyses are complex, detailed and generally are only carried out for research purposes.

A design methodology of shotcrete for application in underground mines was developed for the MHSC namely SIMRAC (SIM 04 02 04) (Joughin, et al., 2012). The methodology is based on the underground monitoring, numerical modelling, laboratory testing and yield line analyses. It summarises the important shotcrete characteristics and rock engineering inputs required for the design of shotcrete. A shotcrete design flow chart is shown in Figure 2-2.

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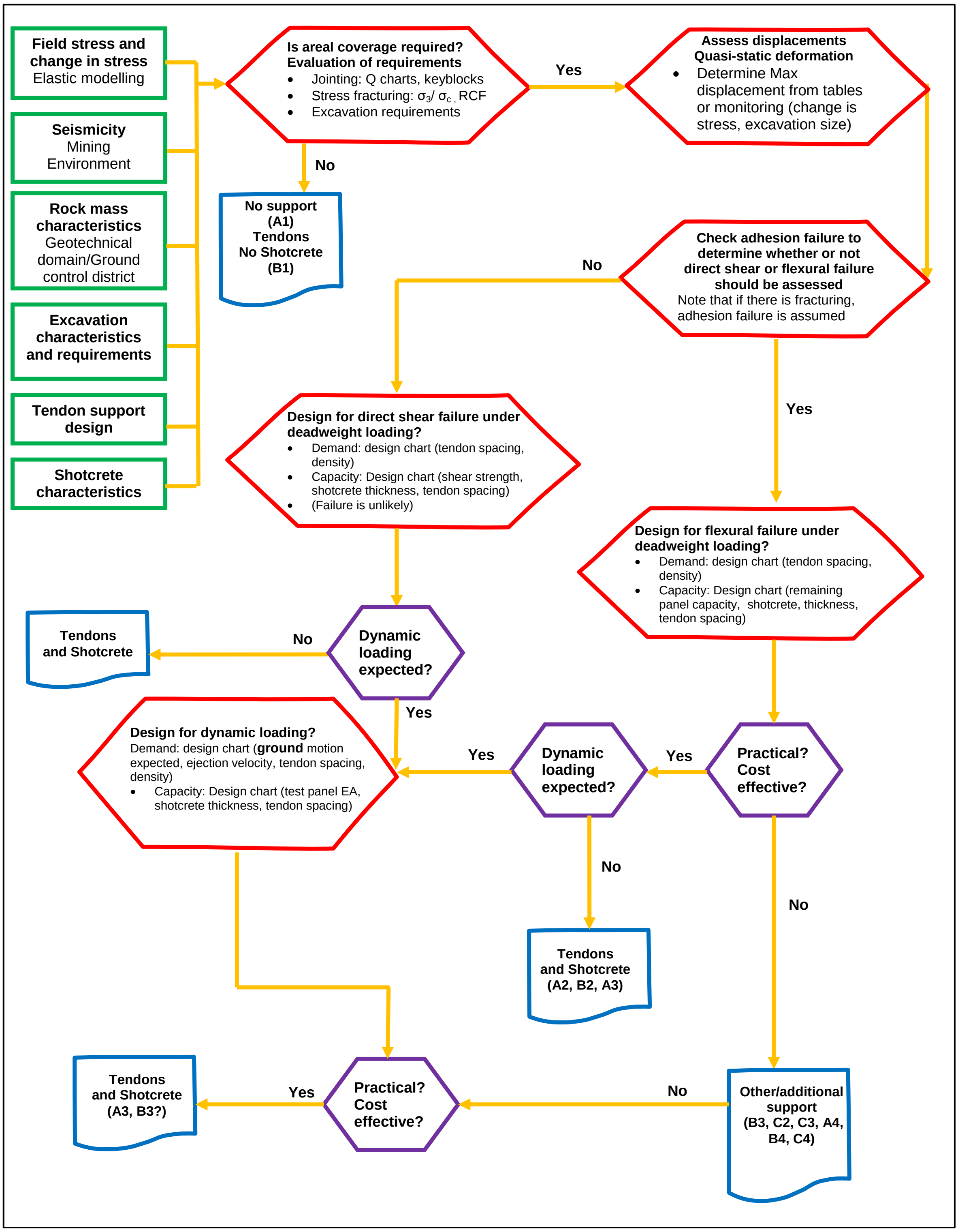

Figure 2-2: Flow chart for shotcrete design in underground mines (Joughin, et al., 2012)

The design of shotcrete is influenced by the mining environment which in turn has a major bearing on the expected shotcrete modes of failure. To fully understand the design and behaviour of shotcrete it is valuable to give an overview of shotcrete failure modes and demand imposed on shotcrete in situ.

2.2.1.(a) Shotcrete failure modes

Barrett and McCreath (1995) have described six basic modes of failure for shotcrete shown in Figure 2-3. *These are*:

i. *Adhesive failure,*
ii. *Direct Shear failure,*
iii. *Compressive failure,*
iv. *Flexural failure,*
v. *Punching shear failure and*
vi. *Tensile failure*

Summaries of the descriptions of the failure modes are given in the subsections that follow.

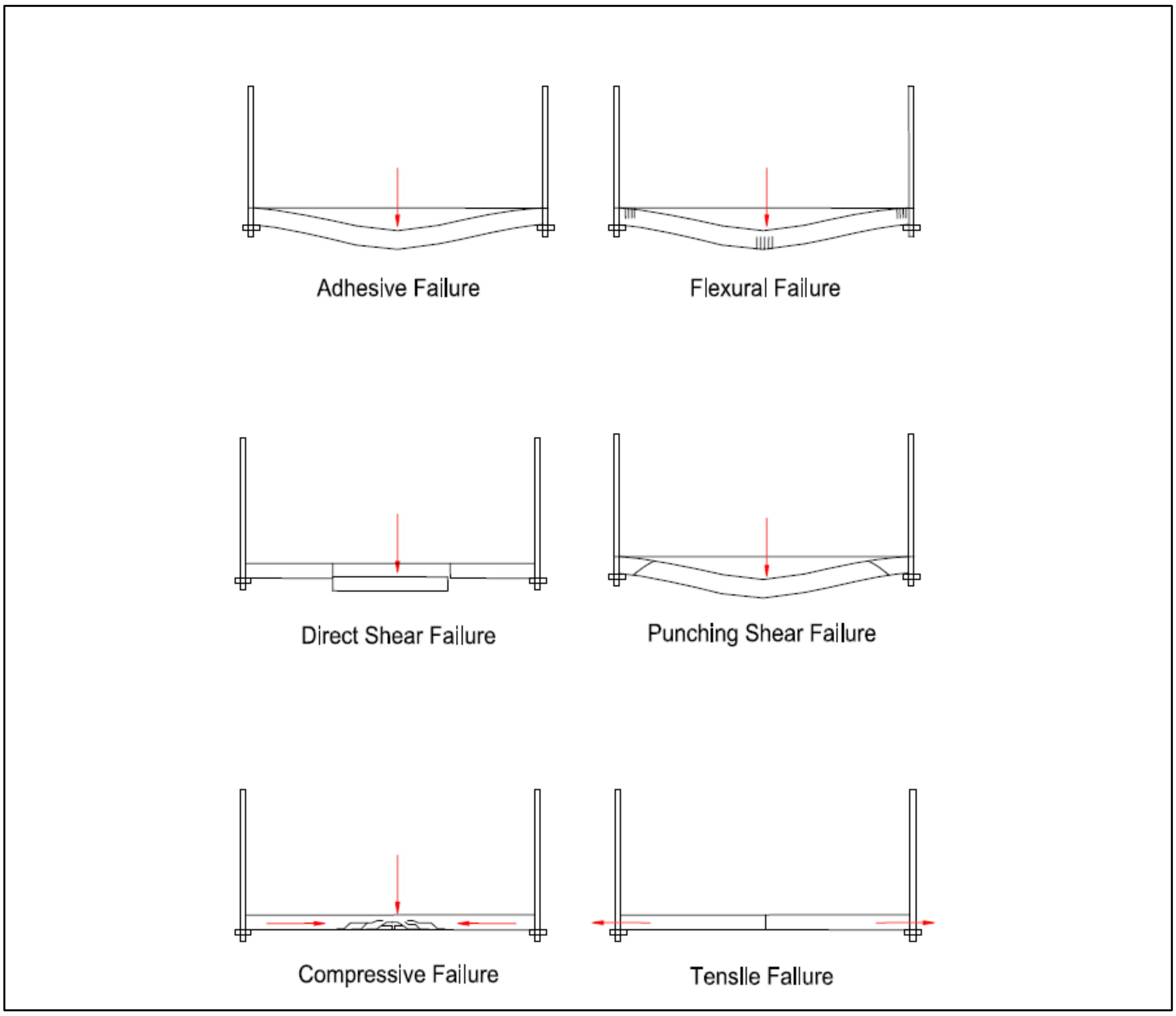

Figure 2-3: Shotcrete failure modes (Barrett and McCreath, 1995).

2.2.1(b) Demand imposed on shotcrete

The demand imposed on the shotcrete is the same regardless of the failure mode. The determination of the demand is explained by Barrett & McCreath, (1995) and is based on a proposed maximum size of a loose rock block between tendons. The size of the block can be estimated by a prism with side angles of 60˚ and a basal area defined by the spacing between tendon shown in Figure 2-4.

The deadweight load (ignoring frictional and block interlock effects) applied to the shotcrete for rectangular tendon spacing can be calculated as:

$$W = \frac{\rho g a b^2 \tan\theta}{6} \qquad \text{(Equation 1.)}$$

where a, b are the larger and smaller tendon spacings,

g is the gravitational acceleration (9.8 m/s^2), ρ is the density of the rock, and

θ represents the side angles of a prism (60˚ for the maximum suggested by Barret and McCreath)

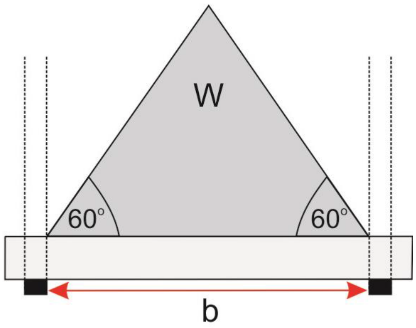

Figure 2-4: Maximum size of loose rock block (Barrett & McCreath, 1995).

2.2.1.a.(i) Adhesive failure

This occurs when there has been a loss of adhesive bond between the shotcrete and the rock surface Figure 2-3. The problem commonly occurs 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). Adhesive failure does not imply shotcrete failure, but simply makes the flexural failure mechanisms kinematically possible. If there is no adhesion failure, then the shotcrete may fail in direct shear or tension.

Adhesion Demand

The demand is simply the load imposed on the shotcrete lining (Barrett & McCreath, 1995), calculated using Equation 1.

Adhesion Capacity

The capacity of a shotcrete lining to resist de-bonding (A_c) for a rectangular pattern as shown in Figure 2-5 is:

$$A_c = 2(a + b)\sigma_{sa} z_a \qquad \text{(Equation 2.)}$$

Where

σ_{sa} *is the adhesive strength of shotcrete*

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. 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 et al., 1981).*

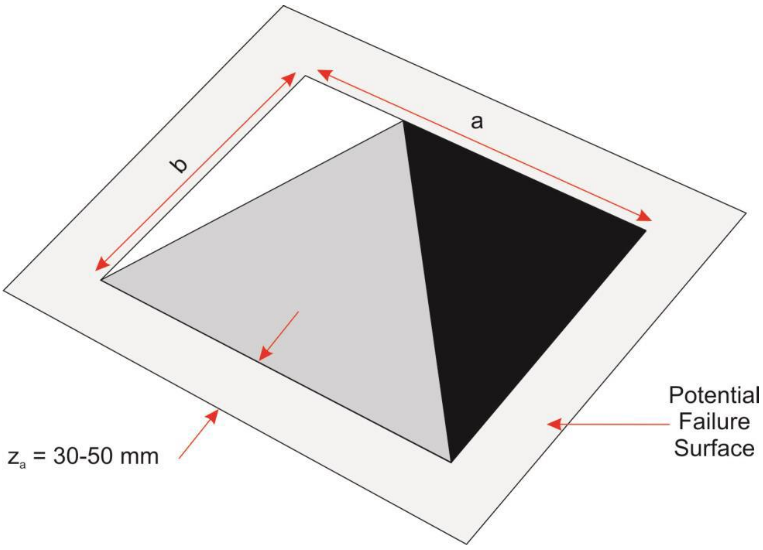

Figure 2-5: Adhesion model modified from Barrett & McCreath, (1995)

2.2.1.a.ii. 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 as shown in Figure 2-3.

Deadweight demand

Barrett and McCreath, (1995) propose that direct shear failure should be determined using the largest block that can be formed between tendons. The demand (T_d) is simply the load imposed on the shotcrete lining using Equation 1:

Shear Capacity

The capacity (T_c) of a shotcrete lining to resist direct shear for a rectangular pattern is:

$$T_c = 2(a + b)\sigma_{ss}h \qquad \text{(Equation 3.)}$$

Where

h — *is the thickness of the shotcrete, and*

σ_{SS} — *is the shear strength of shotcrete in direct shear,*

SABS 0100 - 1, 1992 specifies a minimum design strength of $\sigma_{ss} = 0.75\sqrt{\sigma_{sc}}$ *or 4.75 (anyone which is lesser), where* σ_{sc} *is the compressive strength of the shotcrete.*

The capacity of the shotcrete is a function of its strength and the area over which a load acts. Equations 2 and 3 are similar with differences in the strength function analysed and the area over which the stress acts. Equation 2 analyses the adhesive strength and bond area where the adhesive force acts, as opposed Equation 3 that looks at the shear strength of the TSL that is loaded in the plane of the shotcrete thickness.

2.2.1.a.iii Flexural failure

Once de-bonding of the shotcrete has occurred (adhesive failure), shotcrete can prevent loosening of the rock mass by acting as a slab in bending. Tensile fractures will develop on the outer surface of shotcrete in the centre of the slab where the tensile stress is greatest.

Moment loading demand

Since the rock mass is supported by shotcrete in bending, it is reasonable to calculate the demand imposed on the shotcrete in terms of moment demand. Barrett and McCreath, (1995) proposed the Equation 4 for determination of the moment demand:

$$m_g = \frac{\rho g a^2 b(3b - a)}{96\sqrt{3}(a + b)} \qquad \text{(Equation 4.)}$$

Where

a, b are the larger and smaller tendon spacings respectively;

ρ is the density of the rock; and

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

And, assuming a square pattern:

$$m_g = \frac{\rho g a^3}{96\sqrt{3}} \qquad \textit{(Equation 5.)}$$

Moment capacity

If shotcrete undergoes deflection, it is expected that that it will lose capacity. An equivalent deflection (deformation) can be calculated for European Federation of National Associations Representing producers and applicators of specialist building products for Concrete (EFNARC) and American Society for Testing and Materials (ASTM) C1550 round panels tests. These are panel index tests used for the determination of shotcrete capacity. The equivalent deflection in both EFNARC and ASTM C1550 is determined using the Equation 6:

$$\delta_d = 0.75\frac{\delta_c}{b} \qquad \text{(Equation 6.)}$$

Where

δ_c *Is the maximum displacement which 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 can therefore be carried out to determine ground reaction curves and subsequently ground displacements.*

b *is the minimum support spacing*

Figure 2-6 shows the load deflection graphs determined from ASTM C1550 round panel tests for a range of fibre reinforcement. The remaining load capacity (W_{pc}) can be estimated from this graph. The deadweight capacity is the moment capacity of shotcrete on the wall. This can be determined as follows:

$$m_c = \frac{h^2 W_{pc}}{0.0312} \qquad \text{(Equation 7.)}$$

Where

W_{pc} is the remaining load capacity (kN)

h is the thickness of the applied shotcrete

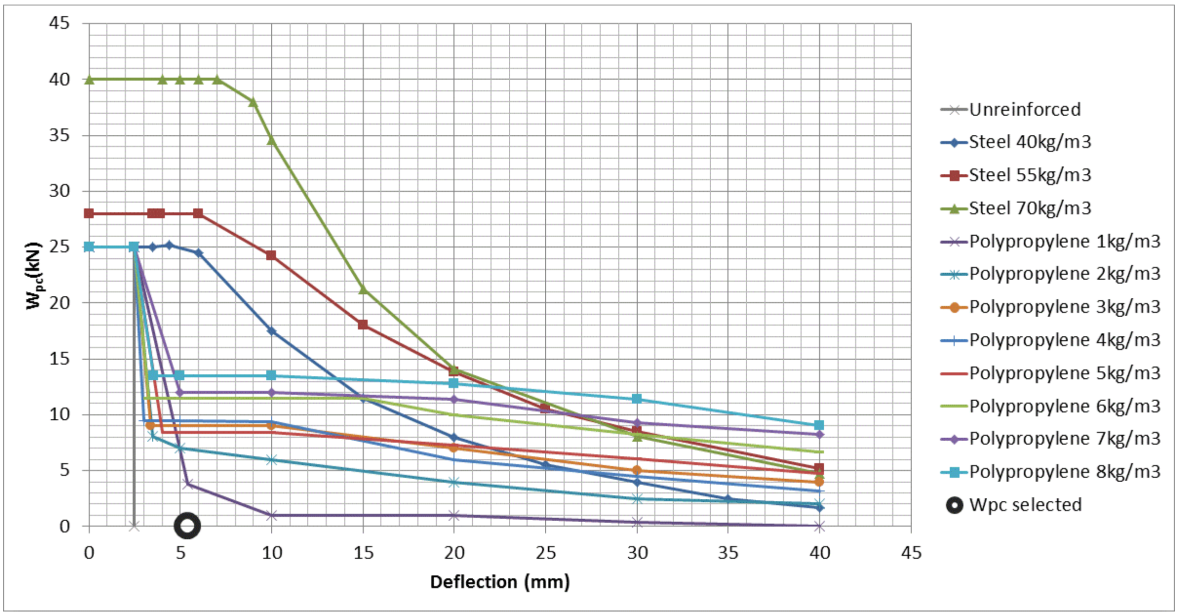

Figure 2-6: Load deflection graphs for ASTM C1550 round panel tests (Joughin, et al., 2012)

2.2.1.a.iv 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 Figure 2-7, but along planes aligned at approximately 45 ° to the shotcrete rock interface, perpendicular to the diagonal tensile stresses in the slab. The shotcrete fails in tension rather than in shear, but it is the shear load that induces diagonal tensile failure (Joughin, et al., 2012).

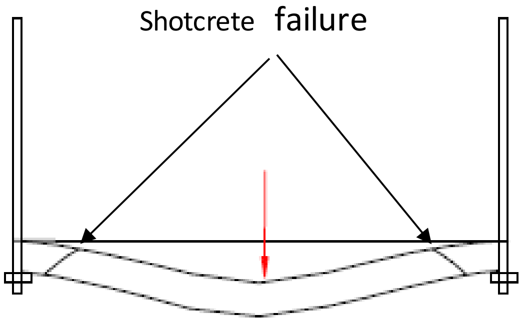

Figure 2-7: Punching shear failure (Barrett & McCreath, 1995)

2.2.1.a.v. 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.

2.2.2. Shotcrete design methodology worked example

During observations at the participating mining operations, data regarding geotechnical characterisation as well as areal support specifications were collected. At Booysendal Platinum shotcrete was used in-stope as areal support where ground conditions dictated. Example 1 below outlines the shotcrete design methodology adopted at Booysendal.

Example 1 - Excavation in blocky rock mass with no stress damage anticipated

Booysendal encountered bad ground in its shallow (approximately 200 m below surface) bord and pillar mining operation. The bords are 8.0 m wide × 2.0 m high and are located in a blocky rock mass requiring areal support. The density of the rock is 3000 kg/m^3 and the excavation is supported by 1.8 m long resin grouted bolts on a 2.0 m x 2.0 m square pattern. No stress changes or dynamic loading are anticipated.

Initial checks

- The ***rock mass*** is blocky and areal support will be required.
- Quasi-static ***displacement*** – not expected.

  The ***weight*** of the pyroxenite rock prism to be supported:

  ρ = 3000 kg/m^3

  g = 9.8 m/s^2

  a = b = 2.0 m

$$A_d = W = \frac{\rho g a b^2}{2\sqrt{3}} = 68\ \mathrm{kN}$$

- Adhesion capacity for low bond strength

$$A_c = 2(a + b)\sigma_{sa} z_a = 400\ \mathrm{kN}$$

  σ_{sa} = 1.0 MPa

  z_a = 50 mm (the hanging wall surfaces were thoroughly cleaned by pressured hoses prior to the application of shotcrete. This ensures good adhesive bond strength.

- Adhesion factor of safety (FoS) = 5.9. This is greater than the desired static FoS of 1.5, therefore design will be for direct shear.

Design for direct shear failure under deadweight loading

- ***Direct shear*** demand (T_d) = W = 68 kN
- Design shear strength: σ_{ss} = 3.4 MPa*
- Direct shear capacity $T_c = 2(a + b)\sigma_{ss}h$
- Required ***thickness***: 25 mm,
- T_c = 2(2+2)*3.4*25 = 680 kN, FoS = 10

Therefore 25 mm thickness of shotcrete ensures a FoS of 10.

* The design shear strength was based on the SABS 0100 - 1, 1992 recommendation that specifies a minimum design strength of $\sigma_{ss} = 0.75\sqrt{\sigma_{sc}}$ or 4.75 (anyone which is lesser), where σ_{sc} is the compressive strength of the shotcrete. The compressive strength of the shotcrete at Booysendal is tested as part of a quality assurance program and was found to average between 20 – 28 MPa after 24 hours. So, a design compressive strength of 20 MPa was used for the purposes of this example.

2.3 Thin Spray-on Liners

TSLs are a relatively new form of rock support and there are polymer based products or water-based materials formed from a combination of cement and sand, or cement only that are applied to the rock with thicknesses that can be as low as 3 mm to 4 mm (Spearing & Hague, 2003). TSLs can be classified as reactive or non-reactive liners. Non-reactive TSLs once applied to the excavation surface gain strength over time as a result of hydration. On the other hand, reactive liners once applied gain strength rapidly due to an exothermic reaction brought on by a catalyst.

TSL advantages are well documented and widely researched. Research by Yilmaz, (2011) summarized these advantages by classifying these advantages into geotechnical and non-geotechnical advantages. The geotechnical advantages have been defined as those that relate to ground stability, enhancing rock reinforcement, support performance and TSL mechanical properties, whereas the non-geotechnical advantages relate to operational, production, logistics-handling, cost of mining-profitability-economic benefits. TSLs are not without their disadvantages and in the same study Yilmaz (2011) made a comprehensive summary of the disadvantages of TSLs. The main disadvantages concern TSL application quality control and equipment requirements; the poor understanding of TSL material properties and technical performance; and the health and safety risks which have been elaborated by (Yilmaz, 2010).

2.3.1 TSL design methodology

Small deformations

Adhesive bond failure of TSL is assumed not to occur in small rock displacements (< 1mm of rock movement) for strongly bonding rigid TSLs. The expected failure modes are either direct shear or diagonal tensile rupture of the liner (Tannant, 2001). These expected failure modes at small displacements are shown in Figure 2-8.

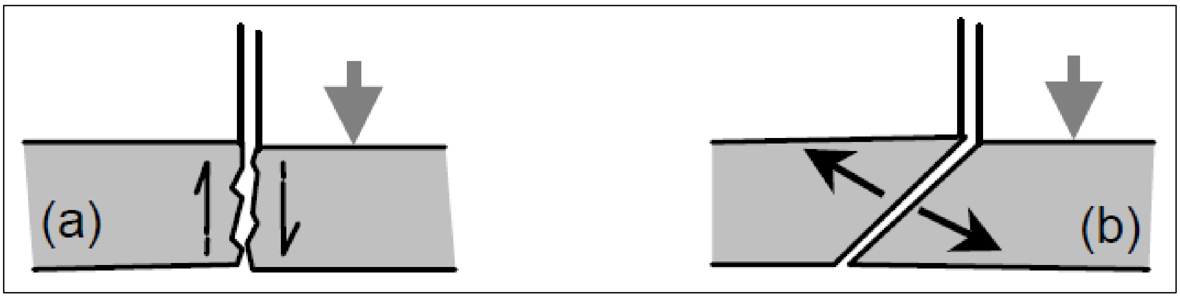

Figure 2-8: Liner failure modes at small block displacements (a) direct shear and (b) diagonal tensile rupture (Tannant, 2001)

The support capacity of the represented failure modes is expressed as force per unit length. It is a function of the liner thickness and strength (shear or tensile strength). Methods to determine the strength of the liners are discussed later in the research. Equation 8 was used to calculate the tensile support capacity of one of the liners from Tumela mine, one of the participating mines, presented in Table 2-1. The 7.5 MPa is for ultra-high strength TSL.

Table 2-1: TSL specifications from Tumela mine (a 'champion' mine) after 28 days

Liner Specification	
Liner thickness (mm)	8.0
Tensile strength (MPa)	7.5
Tensile-bond (adhesive) strength (MPa)	2.5
Shear-bond strength (MPa)	6.5
Material Shear strength (MPa)	17.0

$$F = t \cdot \sigma_t \qquad \text{(Equation 8.)}$$

Where

t *is the thickness of the TSL (mm)*

σ_t *is the tensile strength of the TSL (MPa)*

Using Equation 8, a tensile and shear support capacities of 60 kN/m and 136 kN/m respectively can be derived. Assuming a 1 m × 1 m bolt pattern, the maximum possible block size that can be detached is 1 m^2. Considering rock with a density of 3 000 kg/m^3, the liner could theoretically hold in tension and shear blocks that are 8 m wide and 18 m high in tension respectively. The values of block size that can be supported by 8 mm thick TSL are overly optimistic. TSL specifications listed in

Table 2-1 are for top performing TSLs in each category of strength properties. There is not a single TSL that would address all the specifications listed. Underground loading conditions are irregular and this can further reduce the TSL's support capacity.

Large deformations

Large pull-out tests (>>1mm relative rock movement) done on weak bonding flexible liners have shown block displacements at peak loads that are much greater than the thickness of the TSL (Tannant, 2001). This has demonstrated the deformability and stretching capabilities of TSL before they fail. For stretching to occur there should be some adhesion loss. Consequently, Tannant (2001) provides a failure mechanism for TSL under large deformations, that is, adhesion loss followed by tensile rupture.

The force required to initiate adhesive debonding can be determined using the same logic as Equation 8. However, instead of the TSL thickness, the adhesive bond width is used. The effective bond width dictates the area over which the membrane acts while carrying a tensile load. This parameter is determined from the laboratory through back calculations. The force required to initiate adhesive debonding is 20 kN/m calculated in Equation 9, which is, less than the calculated tensile strength.

$$F = \sigma_a \times w_b \qquad \text{(Equation 9.)}$$

Where

σ_a is the adhesive strength determined from the lab tests; and

w_b is the bond width; for the purposes of this exercise the bond width is approximately equal to the liner thickness (8 mm) which is much smaller than that of shotcrete, a condition required for adhesive failure to occur (Tannant, 2001).

Since the liner's tensile strength is more than the adhesive strength, the liner adhesive bond will progressively fail around the displaced block. Once debonding occurs, whilst resisting the weight of the block, a section of the liner rotates and is loaded in tension as shown in Figure 2-9.

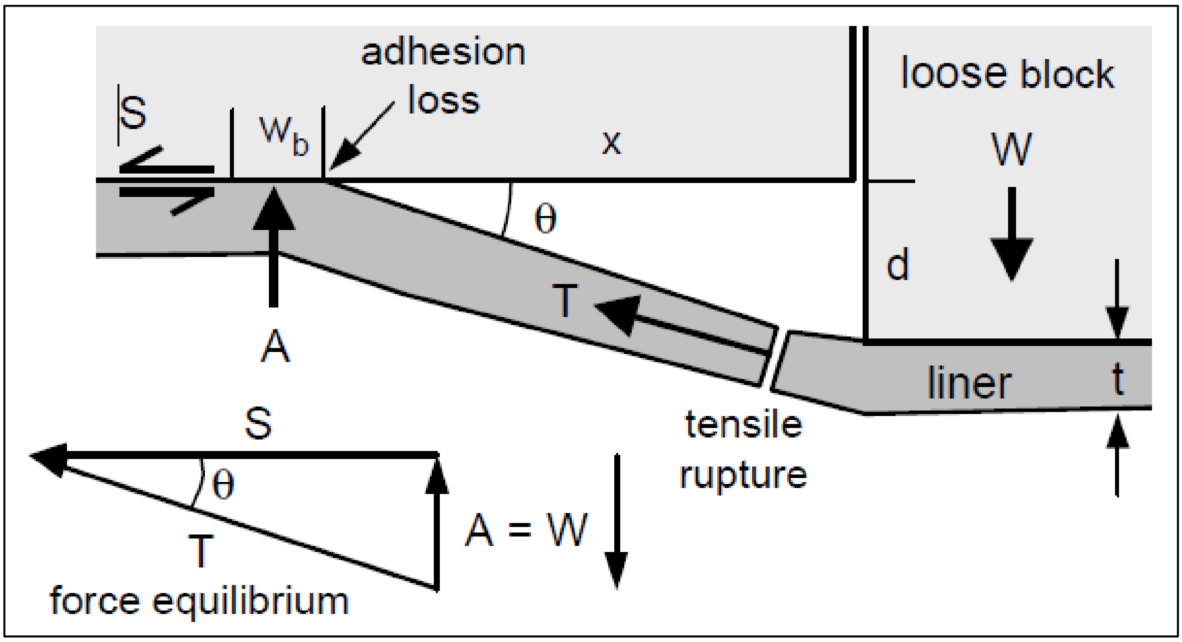

Figure 2-9: Liner adhesion and tensile strength interaction to support a displaced block (Tannant, 2001)

If the block moves sufficiently to cause progressive adhesive failure then debonding will progress away from the edge of the block. This increases the area over which adhesion acts because the perimeter length increases. The area will eventually be large enough resulting in an adhesive force A, which will satisfy the force equilibrium with the weight of the block. The debonded zone width *x* at the moment of tensile failure is determined from Equation 10:

$$A = 4\sigma_a(s + 2x)\,w_b = W \qquad \text{(Equation 10.)}$$

Where

W (kN)	is the weight of the block
w_b (mm)	is the bond width as before (Equation 9)
σ_a (MPa)	adhesive strength acting over the effective bond width
s (m)	is the width of the block.

Tensile stress on the liner is most likely greater near the perimeter of the displaced block. As a consequence the maximum tensile force *T* carried in the plane of the membrane can be determined using Equation 11:

$$T = 4s \cdot \sigma_t \cdot t \qquad \text{(Equation 11.)}$$

A geometric relationship exists between the liner's tensile force and the weight of the block. Knowing the allowable maximum liner tensile force (based on specifications) and block weight estimation, the minimum angle θ can be evaluated using Equation 12:

$$\theta = \arcsin\left(\frac{W}{T}\right) \qquad \text{(Equation 12.)}$$

The angle θ defines the minimum vertical displacement needed to ensure that the block weight *W* is equal to the vertical component of *T.* The equilibrium vertical displacement is calculated using Equation 13:

$$d = x\ tan\ \theta \qquad \text{(Equation 13.)}$$

Based on the model presented in Figure 2-9 at the moment of tensile failure the following relationship holds (Equation 14):

$$\sigma_t \cdot s \cdot t \cdot sin\theta = \sigma_a\ (s + 2x)\ w_b \qquad \text{(Equation 14.)}$$

2.3.2 TSL testing methods developed

Shear – bond strength testing

A steel ring is used to house TSL and rock specimen. The rock core is positioned centrally in the steel-ring as seen in Figure 2-10. The gap between the rock specimen and the steel ring is filled by pouring the TSL. Upon curing the TSL for a predetermined period, the specimen is placed on a base which offers support to the steel-ring and the TSL but not to the rock core. A compressive load is applied on the rock core, displacing the core on the rock / TSL contact towards the void in the support base. The loading and failure of the TSL take place due to shear movement at the rock / TSL contact. Load deformation characteristics are observed until the TSL has failed (Yilmaz, 2007).

Shear movement on the rock-TSL boundary develops shear stress (τ_b) which can be calculated from Equation 15:

$$\tau_b = \frac{F}{\pi D t} \qquad \text{(Equation 15.)}$$

Where;

F: applied force (N)

D: rock core diameter (m)

t : TSL depth or steel-ring height (m)

The stress at the peak force is taken as the shear – bond strength.

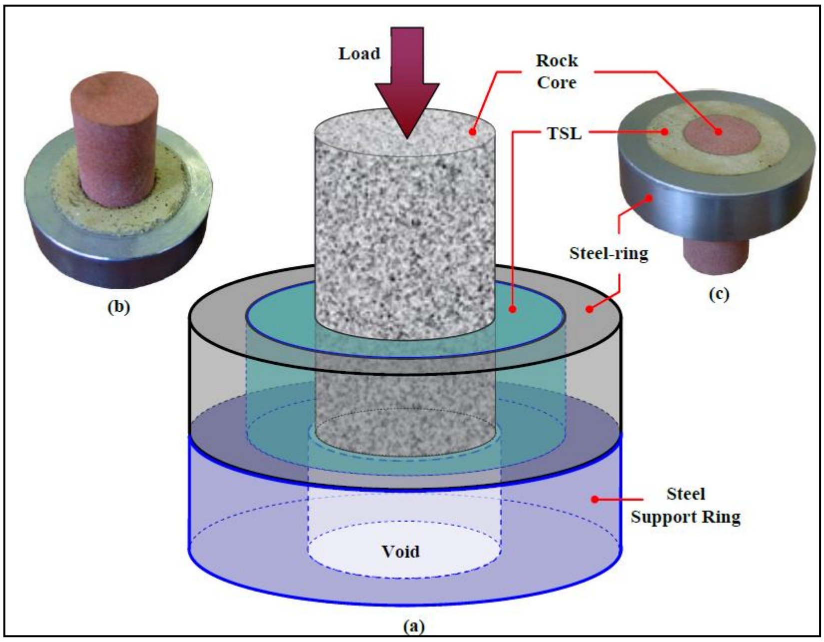

Figure 2-10: Illustration of shear-bond testing, (b) actual specimen top view, (c) actual specimen bottom view (Yilmaz, 2011)

Material shear strength test

TSL is applied inside a steel ring and left to cure. Superimposed holes are drilled on two steel plates. The TSL-steel ring combination is placed in between the steel plates and then clamped. An additional TSL-free steel ring is used as a support base for the clamped specimen assembly. A steel punch of slightly smaller diameter is positioned in the superimposing hole of the top plate and displaced towards the void on the support ring. The test is continued until the ultimate failure load is achieved. The residual shear load level may also be observed. The test apparatus is shown in Figure 2-11 (Yilmaz, 2011).

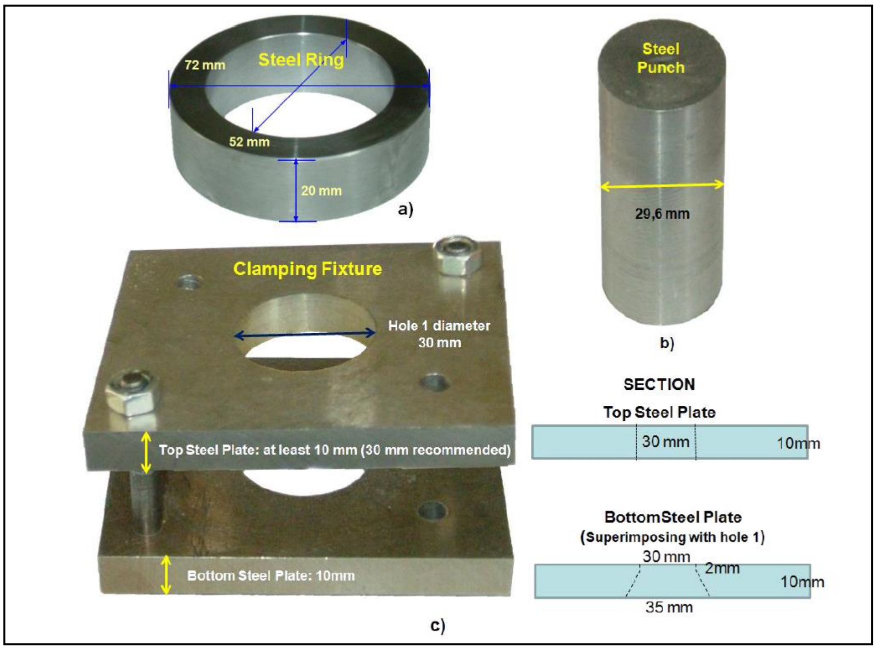

Figure 2-11: Material shear strength apparatus comprising: a) Steel ring, b) Steel punch, and c) Clamping fixture (Yilmaz, 2011).

The shear strength (σ_s) of each test specimen is calculated by dividing the load at failure (F in N) by the area (A in m^2) along which the material fails due to shear as shown in Equation 16 and 17.

$$\sigma_s = \frac{F}{\pi \times d_1 \times t} \qquad \text{(Equation 16.)}$$

Where;

d_1 = steel-punch diameter in m

t = mean thickness of TSL in m

$$\sigma_s = \frac{2F}{\pi \times (d_1 + d_2) \times t} \qquad \text{(Equation 17.)}$$

Where

$\boldsymbol{d_2}$ = bottom plate hole diameter in m

Tensile strength testing

Dog-bone shaped TSL specimens are prepared by pouring into perspex moulds. The specimens cure for a predetermined period under normal laboratory conditions. The specimen is placed in the bottom grip, as shown in Figure 2-12, and tightened while observing the alignment of the long axis of the specimen with the direction of the pull by the help of alignment guide affixed to the stationary frame to prevent any misalignment. Then, the top grip is attached and tightened (Yilmaz, 2010).

The specimen is loaded in tension at a constant loading rate until failure. The failure load is recorded and then the position of failure is inspected for test validity. The dimensions of the failed section are measured with a vernier to calculate the failure area. This measurement can be taken before the test at the narrow section of the specimen. The failure area should also be examined for any anomalous condition such as air bubbles or unmixed TSL lumps to understand the reason for test results that are unexpectedly lower (Yilmaz, 2010).

The calculation of tensile stress takes into account the original cross-sectional area of the narrow section of the specimen. Equation 18 is used for calculating the tensile strength (σ_t):

$$\sigma_t = \frac{F}{A} \qquad \text{(Equation 18.)}$$

Where;

F = load at failure in N

A = original cross-sectional area of the specimen (in m^2) at the narrow section

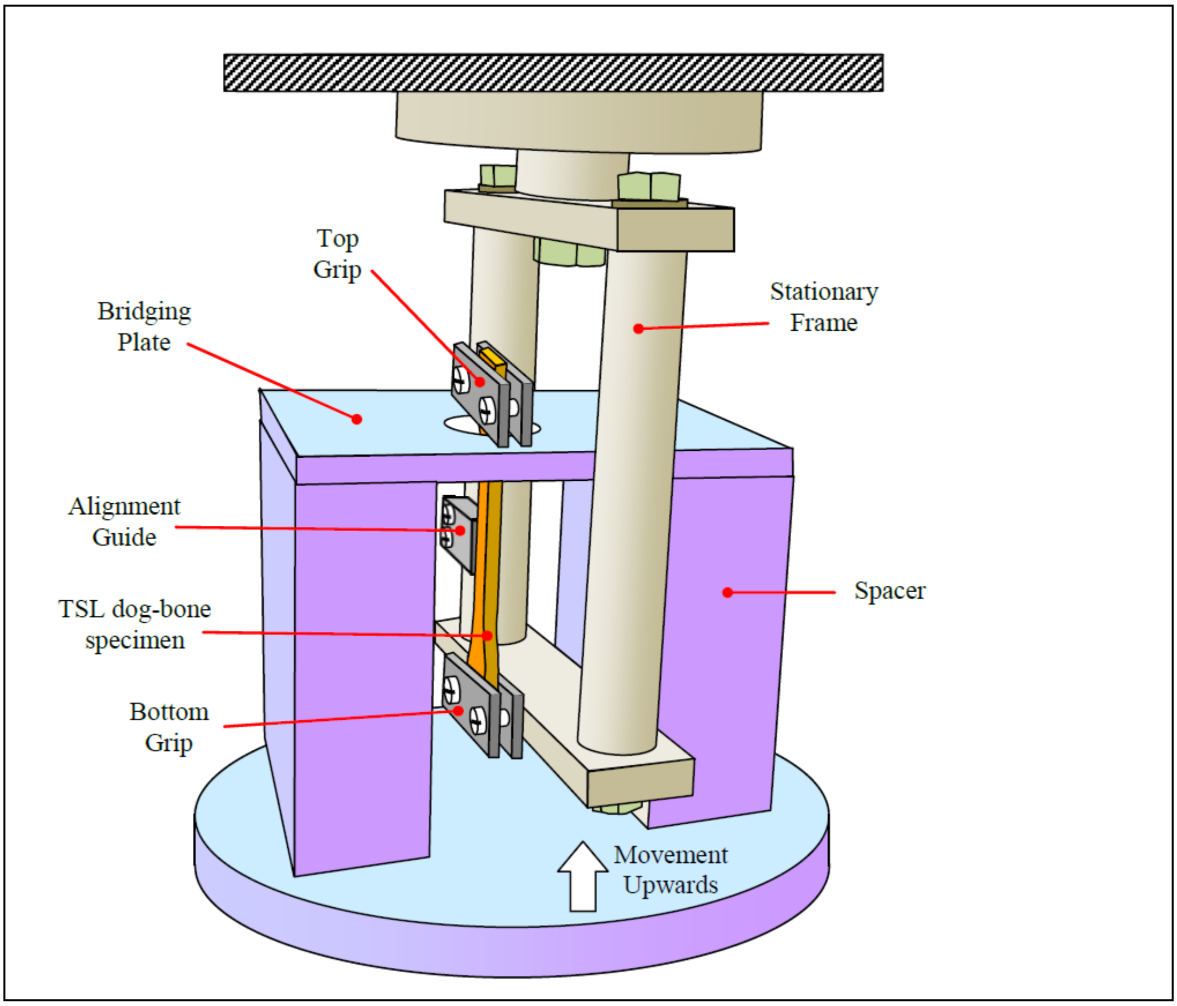

Figure 2-12: Configuration of the tensile strength test assembly to be used in compressive machines (Yilmaz, 2011).

Tensile-bond strength testing

Figure 2-13 shows the specimen used in tensile-bond strength testing where the strength is measured by pulling the steel dolly away from the rock substrate. The failure is expected to take place at the rock-TSL contact for valid testing (Yilmaz, 2011).

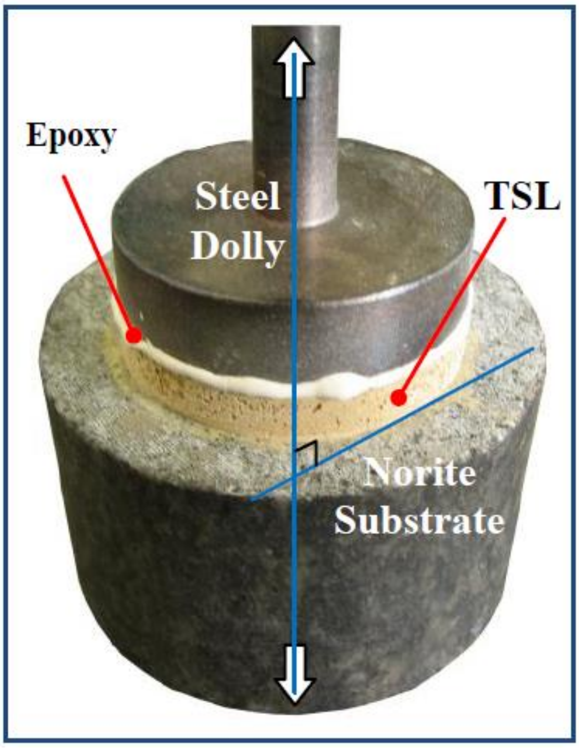

Figure 2-13: Configuration of the specimen in tensile-bond strength testing (Yilmaz, 2011)

The fixture used in tensile-bond strength testing is shown in Figure 2-14. The cured specimen is placed on the bridging plate so that the rock substrate remains on the top as shown in Figure 2-14. The bridging plate has a hole greater than 35 mm in diameter in order to facilitate the passing of the steel dolly. The other end of the dolly is hooked into the groove of the stationary frame that is bolted to the testing machine. None of the ends of the specimen requires clamping. The design of the test setup allows the TSL to be loaded in tension by the upward movement of the bottom platen of the testing machine. The loading direction is perpendicular to the plane of the TSL or substrate. Misalignment of the specimen axis from the direction of pull is prevented by the preparation of flat substrate surfaces and uniform TSL thickness. Then, the steel dolly becomes in line with the axis of the TSL-substrate component after attachment with an epoxy (Yilmaz, 2013).

The loading of TSL is done by load control method at a constant rate. The test continues until the TSL material is detached from the rock substrate while load and testing machine displacement are recorded. The failure load and the diameter of the failure area are noted for calculating the tensile-bond strength. The position of failure is also recorded to explain any anomalous test results (Yilmaz, 2011).

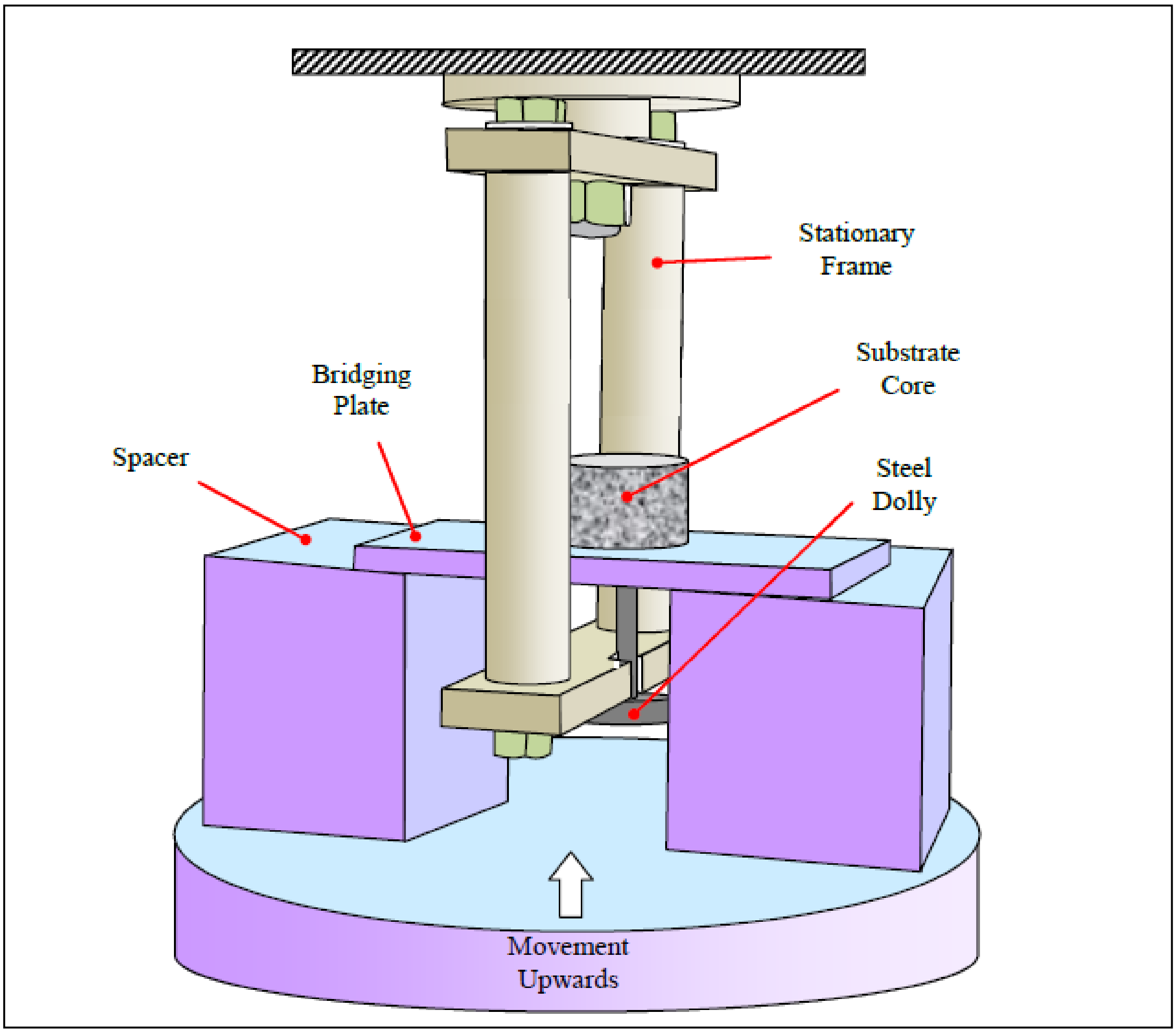

Figure 2-14: Configuration of the tensile-bond strength test assembly to be used in compressive machines (Yilmaz, 2011).

$$\sigma_{tb} = \frac{F}{A} = \frac{F}{\pi \times r^2} \qquad \text{(Equation 19.)}$$

Where

r = TSL radius

2.4 Wire Mesh

Wire mesh is a material that is made from interconnected steel strands. These strands can be woven or welded together. Steel wire mesh technology is widely used for surface rock support in mining and civil engineering. Mesh has been used as ground support in mining for over half a century (Morton, et al., 2008). Mesh has been used largely as a “safety” support system, the purpose of which is to prevent unexpected falls of small rocks (Kaiser, et al., 1996).

Mesh contains rock that detaches between reinforcement support elements, thus taking the form a basket (Stacey, 2001). Thus, mesh is loaded in tension making its tensile properties critical for its performance. The tensile strength of mesh is largely determined by the steel strength it is manufactured from and the thickness or gauge of the wire. Hadjigeorgiou & Potvin, (2011) state that notwithstanding quality control issues and mesh aperture sizes, strand thicknesses determine mesh performance. Typical mesh thicknesses in gauge and standard diameter terms, are shown in Table 2-2. Higher gauge numbers translate to smaller diameters.

Table 2-2: Typical mesh thicknesses expressed in gauge and diameters (Hadjigeorgiou & Potvin, 2011)

Gauge	Diameter(mm)	Strand Strength (MPa)
# 9	3.7 - 3.8	400 - 750
# 6	4.9	
# 4	5.8	

Kaiser, et al., (1996) point out that in high stress conditions mesh provides some confinement to the walls so as to stop the progressive failure processes that lead to unravelling. The stiffness of the mesh elements allows it to resist the displacement. There are three kinds of mesh used as part of an underground support system. These are expanded-metal mesh, chain-link mesh and welded-wire mesh, the latter two being the most commonly used.

Welded wire mesh

Welded wire mesh refers to a metal screen that is constructed from a series of longitudinal and parallel steel wires welded together at grid intersections as illustrated in Figure 2-15. The sizes of the apertures are predetermined depending on the required load bearing capacities. 100 × 100 mm is commonly manufactured, however smaller apertures sizes of 75 × 75 mm or 50 × 50 mm are used if extra load-bearing capacity is required (Hadjigeorgiou & Potvin, 2011).

Figure 2-15: Welded wire mesh (Morton, et al., 2008)

Welded mesh strengths

Tensile strengths of welded mesh strands vary depending on the quality of the steel it is manufactured from. Strand strengths of between 500 MPa and 750 MPa have been quoted by (Jennmar, 2017) an Australian support units supplier. An analysis of data received from steel welded mesh suppliers for the South African mining industry showed that steel strengths ranged from 400 – 600 MPa. # 9 gauge is the most flexible welded wire mesh; however it is more susceptible to blast damage. # 6 is more robust than # 9 gauge mesh therefore it can be used as a substitute; however it is less flexible and more difficult to install.

Installation

The installation of welded mesh can be either mechanised or conventional i.e. by hand. In a mechanised mine, bolting machinery is used whereas in a conventional mine, handheld drills are used. Larger diameter welded mesh is stiffer. Hadjigeorgiou & Potvin, (2011) noted that it is very difficult to install it tightly to the rock surface. As a consequence the stiff support capability of the welded mesh can be lost.

Failure analysis

Morton, et al., (2008) observed three different modes of failure for welded wire mesh which are a measure of the mesh quality as seen Figure 2-16:

- *tensile wire failure due to excessive tensile load*
- *shear failure at the welded joint*
- *failure of the wire through the heat affected zone (HAZ) due to excessive loading and heat.*

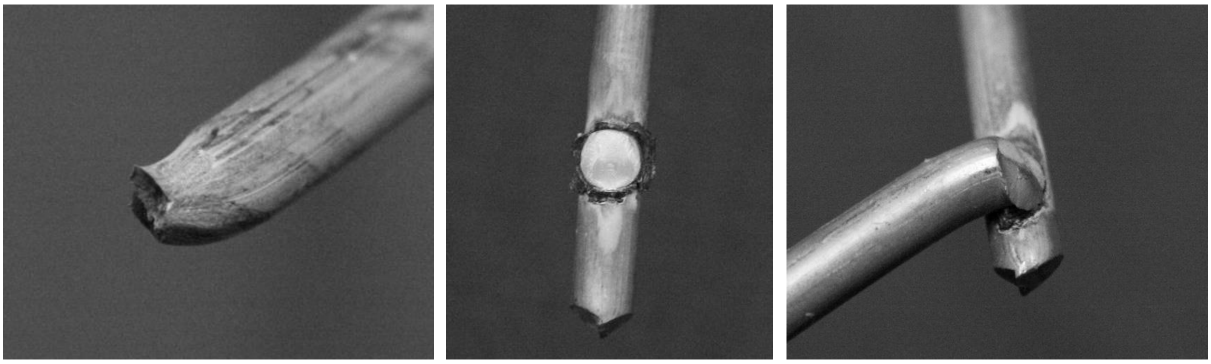

Figure 2-16: Welded wire failure modes; from left to right - tensile wire failure, shear weld failure and failure of the wire through the HAZ (Morton, et al., 2008)

Chain – link mesh (diamond mesh)

Chain link mesh is sometimes referred to as diamond mesh due to the grid patterns. Steel wire strands are shaped in a zigzag manner and then are woven together to form a diamond pattern, illustrated in Figure 2-17. The typical aperture and strand diameters found in the South African mining industry for chain link mesh are 75 mm x 3.2 mm, 100 mm x 3.2 mm and 100 mm x 4 mm (Stacey, 2001).

Chain-link mesh strengths

The tensile strength of chain link depends on the strength of the steel as well as the environment it would be used in. Yield tensile strengths of between 500 MPa and 700 MPa have been reported by suppliers (Jennmar, 2017). It is possible to have mesh with tensile strengths of up to 1.8 GPa (Player, et al., 2008) and this kind of mesh is normally referred to as ultra-strength mesh.

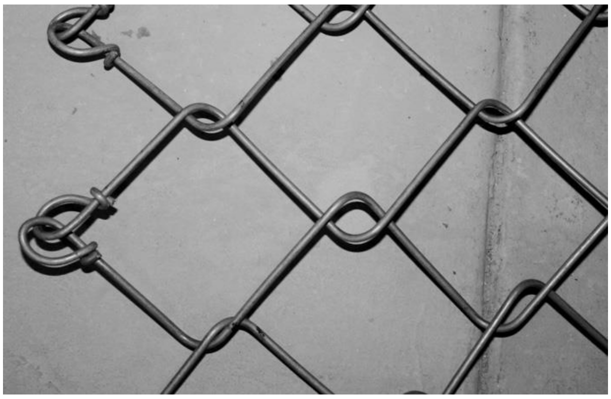

Figure 2-17: Chain link mesh (Morton, et al., 2008)

Installation

Chain link is less rigid than welded mesh and this has been identified as both a disadvantage and an advantage according to the intended purpose. The flexibility of chain link mesh can result in difficult handling and installation, particularly when mechanised equipment is used (Hadjigeorgiou & Potvin, 2011). Hadjigeorgiou & Potvin, (2011) state that South African and South American mines are relatively less mechanised and therefore the flexibility of the mesh aids in the installation process. The excessive deformation capabilities of the mesh, as well as its failure to carry load when one of the strands is broken are some of the major drawbacks of chain link mesh.

2.4.1 Mesh design

Mesh design capacities and parameters are developed based on laboratory test results. The tests should evaluate the effects of bolt spacing, wire diameters, and bolt plate loads on the capacity and displacement of the mesh. Numerical modelling can be used to do parametric evaluation and interpretation of the mesh design and capacities.

Gadde et al. (2006) have used non-linear numerical modelling to evaluate the behaviour of mesh. They utilised the beam and pile elements in the software FLAC3D to simulate the mesh. In the modelling, it was assumed that there was no slippage at the rock-wire-plate interface and as such the mesh was fixed at the location of the bolt face plates. The mesh was modelled using the elastic-perfectly plastic material behaviour on large strain mode to get the load capacity and stiffness of the mesh. Ultimately the test and modelling results can then be used to develop a design criterion for the mesh.

Test results can be presented graphically in the form of a load – displacement curve as shown in Figure 2-18. The components of the curve are described as follows:

- Peak load: maximum load carried by the mesh prior to a significant drop in load
- Design load: maximum load prior to a significant decrease in the stiffness of the mesh
- Mesh stiffness is determined as the slope from a point at 20 % of the design load to the design load. The stiffness of the mesh can be calculated using Equation 20:

$$K_m = \left(\frac{L_d - L_{20}}{D_d - D_{20}}\right) \qquad \text{(Equation 20.)}$$

Where;

K_m	is mesh stiffness
L_d	design load
L_{20}	20% of the design load
D_d	Design load displacement
D_{20}	Displacement at 20 % of design load

Displacement offset, D_o: defined as the intersection of the line used to calculate the stiffness and the x-axis

For the purposes of mesh design and evaluation, the design load is used instead of the peak load. This is as a result of reduction in mesh stiffness, an indication that the mesh performance is either dominated by slippage at the bearing plates or there have been wire breakages.

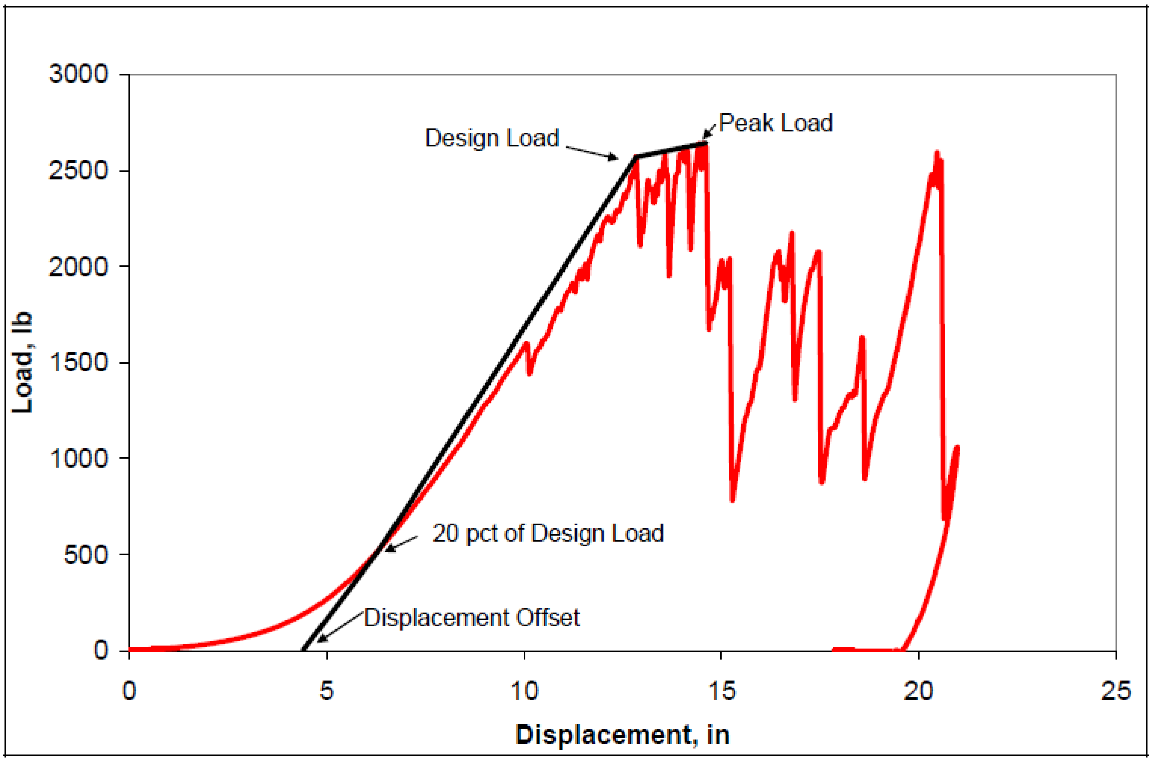

Figure 2-18: Load - displacement curve for a test showing parameters used to evaluate mesh performance (Dolinar, 2009)

The capacity and performance of mesh was found to depend on the load as well as frictional conditions at bolt face plates (Dolinar, 2009). Due to the irregularity of rock surfaces in underground mining situations, parameters that affect mesh capacity and performance are highly variable. There are different degrees of fixity and slippage (boundary conditions) of the mesh at the bolt face plates. When developing estimates of the in-situ mesh performance for the purposes of design based on lab tests, it is reasonable to average the results of the different face plate loading conditions.

Knowing the expected ground displacement the average load imposed on the mesh can be determined using the Equation 21:

$$L_m = K_m \times D_m + L_o \qquad \text{(Equation 21.)}$$

Where

D_m = Mesh displacement

K_m = Mesh stiffness

L_o = Offset load (explained below)

A linear load – displacement curve which can be plotted using the average design load, 20 % design load and the load at the displacement offset together with the corresponding displacements. The offset introduced in the equation is the intercept of the linear load – displacement curve with the load axis, the offset load is negative. A typical linear load – displacement curve for different wire diameters is shown in Figure 2-19.

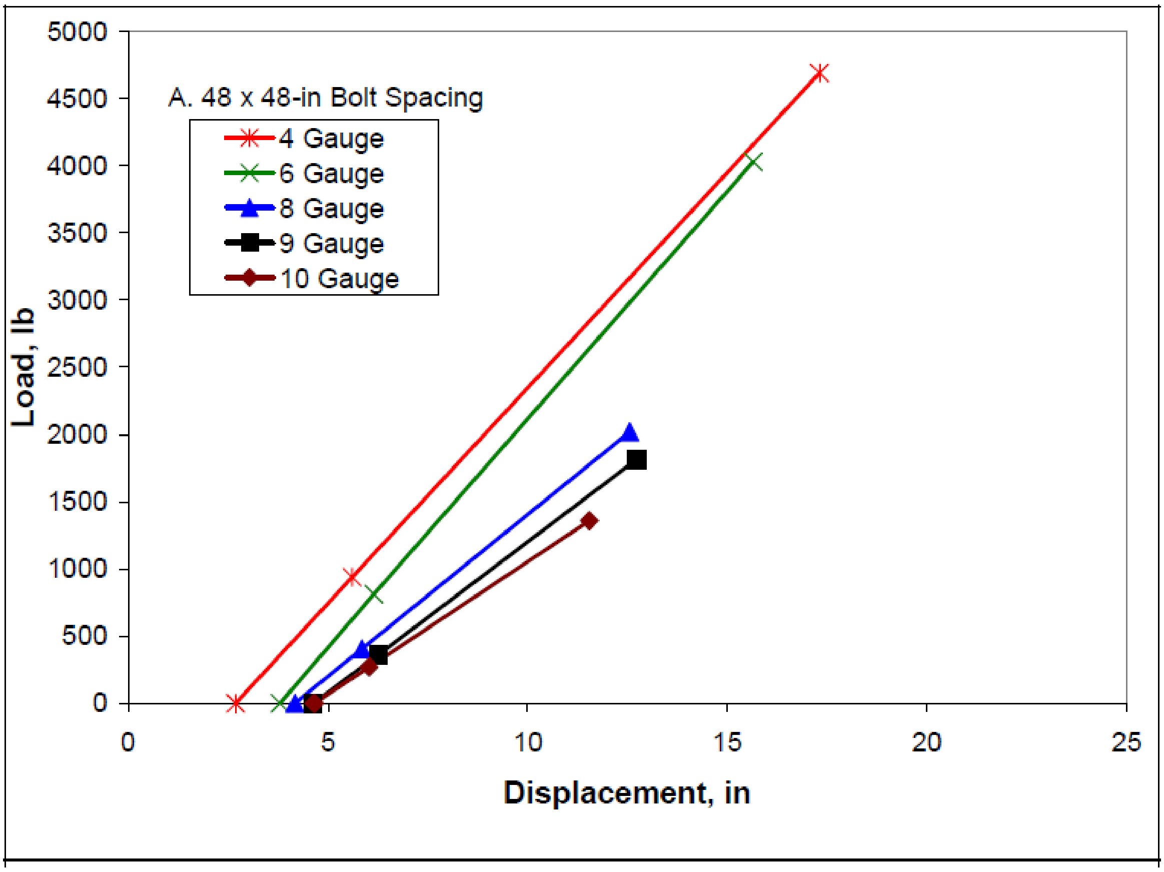

Figure 2-19: Linear load-displacement curves for different wire diameters (Dolinar, 2009)

Mining in deep mines or in relatively stressed ground but not necessarily deep (for example in South African platinum mines), there is a likelihood of seismic events as well as dynamic loading of support.

The magnitude, location and the number of seismic events cannot be predicted, as a consequence making the design of dynamic support an arduous endeavour. The demand imposed on the support system by rockbursts is expressed in terms of kinetic energy (Equation 22), which is a function of the ejected rock mass and its velocity. The ejection velocity is dependent on the magnitude of the event and the attenuation of the peak particle velocity (ppv) with distance from the source of the event (Potvin, et al., 2010). For the purposes of forward analysis and design to assess demand, the designer has to assume an event location and magnitude. A method to determine the relationship between ppv – magnitude – distance was presented by Kaiser et al. (1996). However; in South Africa 3 m/s is the commonly used ejection velocity for rocks subjected to dynamic movement for design purposes (Ortlepp & Stacey, 1997).

$$Energy = \frac{1}{2}mv^2 + mgh \qquad \text{(Equation 22.)}$$

Where

m is the mass of the ejected rock (kg)

v is the velocity of the ejected rock (m / s)

g is the acceleration due to gravity (m / s²)

h is the distance travelled by ejected rock mass (m)

When a force causes a body to be displaced, it is said to be doing work. The basic work relationship is expressed as a product of force multiplied by displacement. Under special cases such as a constant force, work done on a system is determined as the area under the force – displacement curve. The force acting on a system may vary in both magnitude and direction, as well as the path followed by the force. All these issues can be taken into account by defining work as an integral of force and displacement. This definition amounts to an infinite sum of the products of the component of force along the path times the corresponding path length (Feynman, et al., 1964).

During the simulation of the behaviour of mesh under a rockburst situation in the laboratory, a load-displacement curve can be derived, and it is characterised by a varying force and path due to the resistance offered by the mesh. To estimate the work done on the system, the area under the force-displacement curve is calculated. Several numerical integration methods can be used to approximate the area under the curve, such as the trapezoidal rule and the Simpson’s method. The energy capacity of the mesh is then compared to the demand calculated in Equation 22 to give factor of safety (Potvin, et al., 2010).

2.4.2 Western Australia School of Mines (WASM) testing programmes

Mesh can be used to provide areal coverage for both dynamic and static loading conditions. The testing of mesh for both loading conditions is different and as a consequence the Western Australia School of Mines (WASM) constructed rigs to perform both dynamic and static tests. The testing facilities for both systems are described in the following subsections.

Static test facility

Morton et al., (2008) have described the WASM static test facility and shown in Figure 2-20 and Figure 2-21. The test rig comprises two steel frames; a lower frame that acts the structural support for the samples to be tested and upper frame which provides loading reactions. The loading frame is restrained within a stiff frame that rests on the support frame. The restraint systems consist of threaded bar, eye nuts and D – shackles passing through the perimeter frame.

A screw jack which is mounted on a reaction frame is driven at constant speeds allowing for large displacements to be imposed on the mesh. Load is applied to the mesh through a spherical seat to a 300 mm square thick steel plate. The force is measured using a 50 tonne load cell.

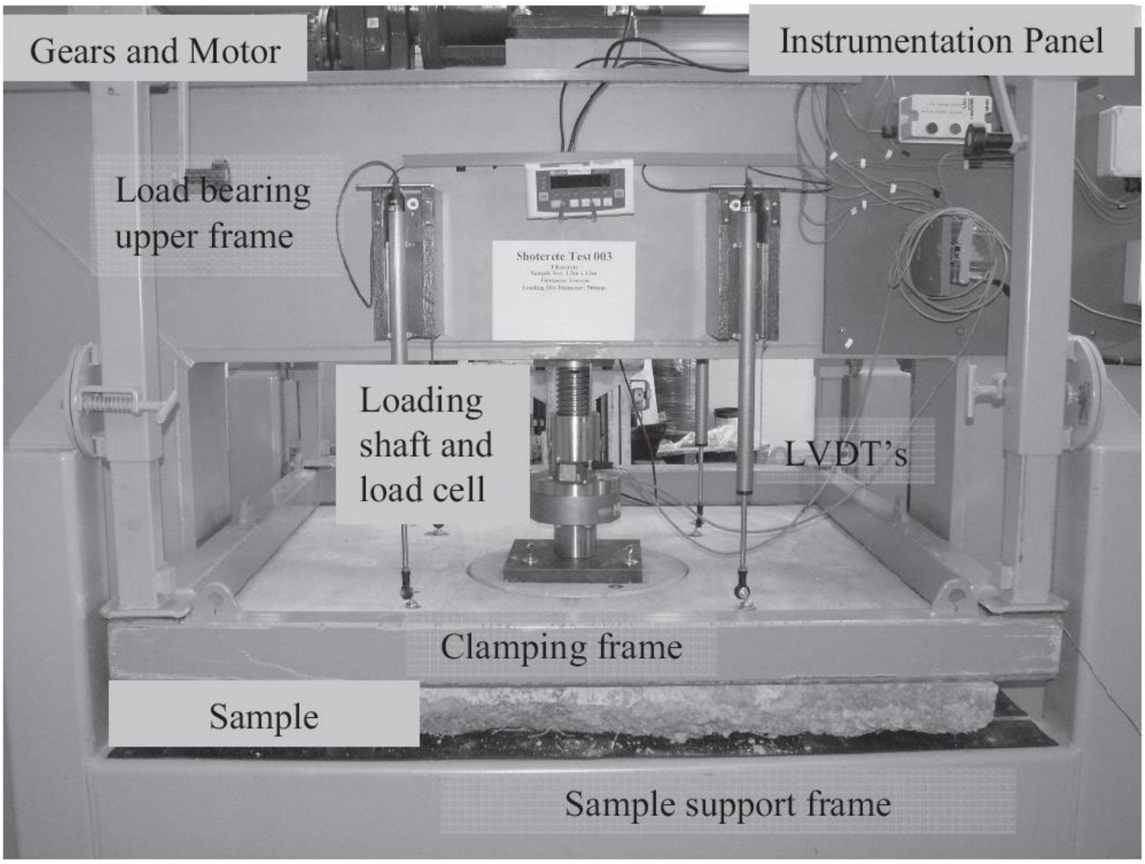

Figure 2-20: Static test facility (Player, et al., 2008)

Figure 2-21: Boundary restraint system (Player, et al., 2008)

Dynamic test facility

Player et al.(2004), have described the WASM dynamic test facility shown in Figure 2-22. The test facility consists of a drop beam positioned between four guide rails. Samples are loaded using the momentum transfer concept. A frame, to support the mesh, is bolted to the drop beam. The mesh is held in place using threaded bar, shackles and eye bolts in the same configuration as the standard static test arrangement. The loading mass consists of a pyramid shaped bag filled with a known mass of steel balls (0.5 or 1 tonne). The loading area of the bag is 650 mm x 650 mm. A wooden prop is placed between the loading mass and the drop beam to prevent the mass "floating" during the initial free fall period. The drop beam and attached mesh frame assembly are dropped from a specific height to generate dynamic loading on the mesh sample. Computer software, advanced instrumentation and a high speed video camera are used to record the test data.

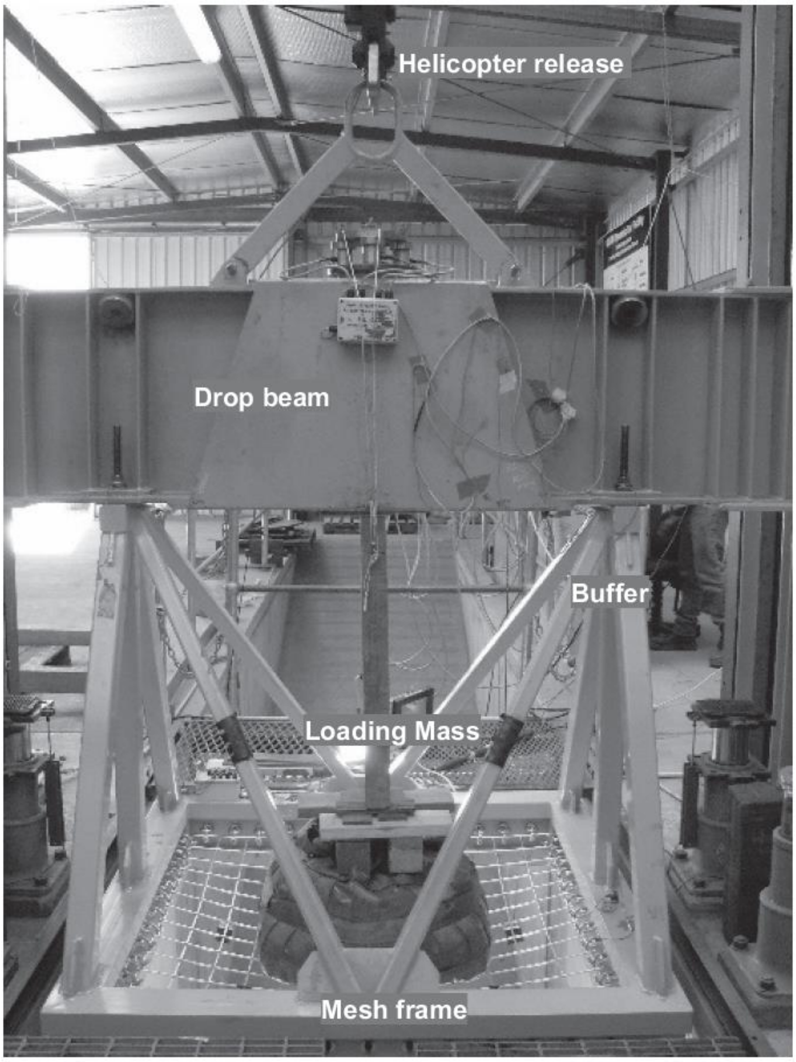

Figure 2-22: Dynamic test facility (Player, et al., 2004)

2.4.3 Ortlepp and Stacey tests

A large scale laboratory test facility was developed to simulate dynamic loading of containment support (Ortlepp & Stacey, 1997). This was done to determine the performance characteristics of containment support elements under dynamic loading conditions. In a rockburst situation, the loading imposed on the support is in the form a violent impact. To approximate realistic field conditions as much as possible a drop weight was used to represent the impact. The complete dynamic loading setup consisted of rock bolts 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.

Figure 2-23 illustrates the testing facility and its features are described below:

- For wire mesh or wire mesh and lacing, a 2 m × 2 m area of mesh was supported by four rockbolts spaced 1 m apart.
- For wire mesh reinforced shotcrete or FRS, the size of the panels allowed for 300 mm overlap outside the 1.0 m × 1.0 m rockbolt panel. The tested panels were therefore 1.6 m × 1.6 m.
- The load distribution system consist of packed concrete blocks in direct contact with the containment support to simulate the rock mass, and a pyramid of steel – clad, load-distribution elements above this to distribute imposed load to the whole of the central support surface.
- Edges of the test panel were constrained to only have limited movement downwards and inwards.
- The test rig was designed to have impact loading velocities of up to approximately 8 m/s and energy input up to approximately 70 kJ.
- The containment support is supported by 22 mm diameter cone-bolts
- A traversing load suspension frame and the drop weight.

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

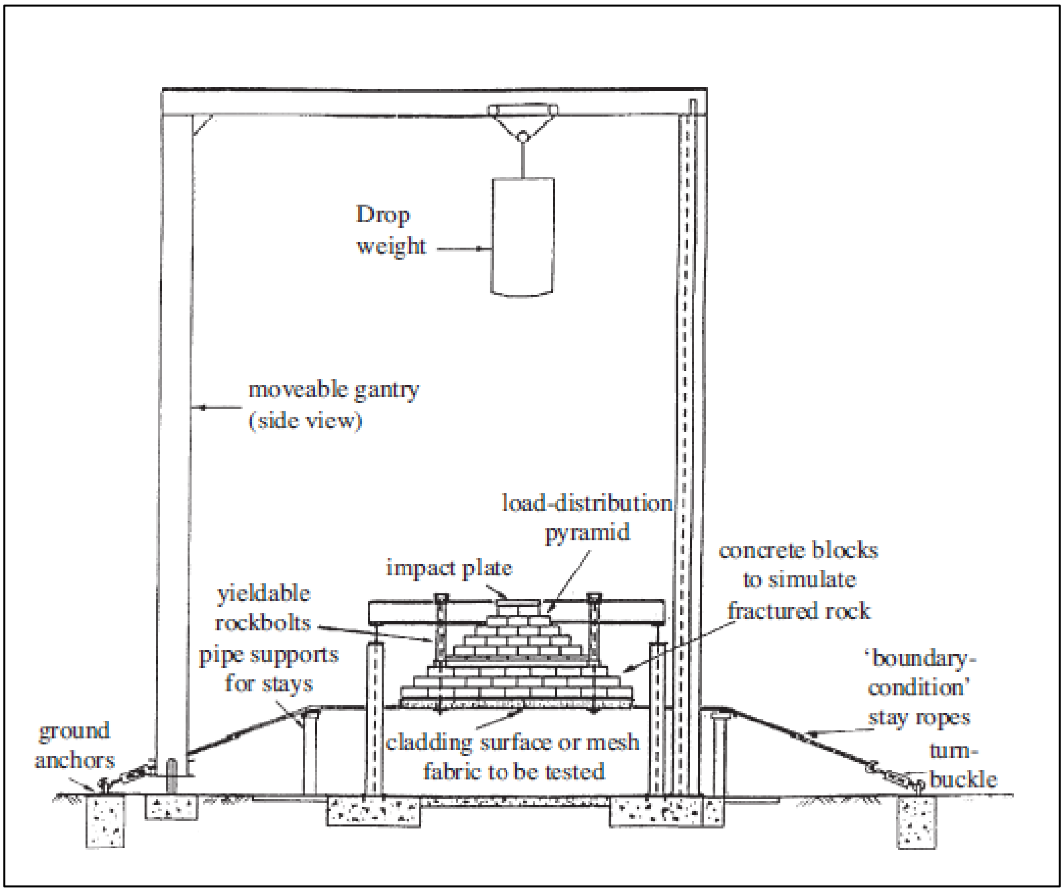

Figure 2-23: Dynamic test loading facility (Stacey & Ortlepp, 1999)

3. UNDERGROUND OBSERVATIONS

The interaction of areal support and the rock surface to which it is applied can vary considerably when laboratory (ideal) conditions are compared to underground (real) conditions. To fully appreciate behaviour and performance of areal support it was deemed beneficial to carry out underground observations.

3.1. Objective of underground observations

The observations aim at investigating the existence of permanent areal support technologies including clear visual records of the successes and failures thereof. The outcome of the underground observations together with the expected performance of the support system and outcome of the literature review will be used as key inputs for the development of the methodology for the selection of areal support and the ranking tool.

3.2. Observations sites

10 separate sites shown in Table 3-1 were identified and subsequently observations were carried out. Each of the site establishments, geological reasons for using the support units and the different support units used are summarized in the subsections that follow. Detailed discussions of each of the sites and the application of the assessment tool carried out by the writer can be viewed in Appendix A.

Table 3-1: Support systems evaluated at participating mines

Site number	Support system description	Participating mines
1	Steel rope mesh (netting) with tendons [high stope width]	Bambanani East Mine, Harmony Gold, Welkom
2	Steel rope mesh (netting) no tendons [low stope width]	Bambanani East Mine, Harmony Gold, Welkom
3	Shotcrete	Booysendal Mine, Northam Platinum
4	Shotcrete	Booysendal Mine, Northam Platinum
5	Steel welded mesh with hydrabolts (friction tendons)	Ikamva shaft (Sibanye Gold Kloof 4 Shaft)
6	Steel welded mesh and split sets (friction tendons)	Tau Tona, AngloGold Ashanti
7	TSL	Two Rivers Platinum (ARM-Impala JV)
8	TSL	Tumela Mine, Anglo Platinum
9	Chainlink mesh and mechanical end anchor bolts	Lonmin plc Karee 4 Belt
10	Steel welded mesh and cable anchors	Dishaba Mine, Anglo Platinum

3.2.1. Bambanani East Mine, Harmony Gold, Welkom

Bambanani Mine is situated in the Free State portion of the Witwatersrand Basin, between Welkom and Virginia. The rest of the mine having ceased, mining is now taking place in the shaft pillar which is traditionally a high risk mining zone characterized by frequent seismic events and intensely stress fractured hanging wall. The main orebody exploited is the Basal Reef which is overlain by weak Khaki Shales. The Khaki Shale is intimately associated with the overlying Waxy Brown Leader Quartzites. The facies changes act as weak planes and these, together with the fact that mining is in the shaft pillar, result in difficult ground control.

When a seismic event occurs, shake-down of the stress fractured hanging wall usually occurs along the Khaki Shale bedding planes and this poses risks of accidents. Therefore, there is a need to provide ground support to mitigate and control the risks posed by fall of ground. The support system used at Bambanani comprises of friction based tendons (in low stoping cuts – refer to appendices for the different stoping cuts), timber packs, and steel safety nets as the permanent areal support units. Rapid yielding hydraulic props are also used as part of the support system with each face advance albeit being temporary support units.

3.2.2. Booysendal Mine, Northam Platinum- UG2

Booysendal, situated on the eastern limb of the Bushveld Igneous complex, is approximately 37 km from Steelpoort Mpumalanga. The mine exploits platinum ore from the Upper Group 2 (UG2) reef and is currently developing to mine the Merensky reef. The mine is a largely mechanised shallow depth room-and-pillar operation. Primary ground support consists mainly of grouted rebars installed by mechanised roof bolters and where geotechnical conditions dictate, cable anchors are installed as well as in - stope shotcrete. The shotcrete is applied using hand-held methods (pneumatic spray pumps). The shotcrete may be applied up to the face or lagging behind the face as required. However, routine application up to the face is not common practice. As a result, blast-on behaviour of the shotcrete is only occasionally visible and was not available for observation during the site visit.

Observations were carried out at three respective stoping sites for varying ground conditions in the UG2 operation where shotcrete was undertaken, viz "normal", "pothole" and "dyke" conditions. Additional observations were carried out at tip locations for long-term excavation support, classed as normal ground conditions.

Two shotcrete-based standards are applied according to ground conditions, viz with cables and without cables. The application of shotcrete was on an ad hoc basis i.e. no set standard except for long-term excavations (tip areas).

Ground condition assessment using a quantifiable method is applied using rock mass classification (RMR, Q). However, systematic support design verification using joint set mapping and structural analysis such as JBlock is not evident. Only significant structures are mapped by geologists (geotechnicians) and plotted on plans.

3.2.3. Booysendal Mine, Northam Platinum- Merensky

Observations were carried out in the Merensky decline operation where shotcrete application was done due to the long-term nature of the excavation. These observations are described in the worksheets. Borehole camera surveys assist to identify gravitational block failure hazards and guide installation of secondary support. The use of shotcrete together with mesh was installed at the portal entrance of the decline and in the portal walls. However, this installation had been completed at the time of the site visit. The initial plan for the project, i.e. to observe mesh with shotcrete installation, therefore had to be adapted for shotcrete only. To distinguish between shotcrete application in the Merensky decline from shotcrete application in the UG2 stoping horizon, observations focused on the different types of challenges in a high excavation height development versus limited stope height (UG2). The differences are discussed in the Appendix A.

3.2.4. Ikamva shaft (Sibanye Gold Mines' Kloof 4 Shaft)

Ikamva shaft formerly known as Kloof 4 shaft is a conventional deep to ultra deep gold mine. Kloof is located on the West Wits Line, which forms the Far West Rand of the Witwatersrand basin. The immediate hanging wall comprises komatitic volcanic lavas which are often sheared and display cooling contraction joints, in addition to quartz and calcite veining, resulting in localized poor hanging wall conditions. The Westonaria Formation lava (WAF) is the most common rock type in the hanging wall. However, the VCR quarzite formation is also encountered.

The hanging wall conditions led to localized ground control problems as blocks of broken hanging wall dislodged frequently. As a result, permanent areal support was required particularly in gullies as they serve as long term access ways. Plastic mesh (New Concept Mining (NCM) - Gabion mesh) was therefore installed as permanent areal support in the gullies. However, due to poor performance of the plastic mesh the areal support was changed to welded steel mesh. At the time of the investigation the practice of steel weld mesh had been in effect for approximately two weeks. Installation of the mesh largely started as follow behind protection for older gullies progressing up to 8 m from the blasted face. This limited the investigation of the mechanical resistance of the mesh against equipment and blast action.

3.2.5. Tau Tona, AngloGold Ashanti

Tau Tona is situated on the West Wits line, south of Carletonville, about 70 km south-west of Johannesburg. Tau Tona is a largely conventional ultra-deep operation using scattered sequential grid to exploit gold resources. The immediate hanging wall is composed of siliceous quartzite averaging 2 m thick which is very competent. The quarzite is overlain by a chloritoid shale (Green Bar) approximately 2 m thick, which has be soapy feel and a poor cohesive strength. Where the immediate hanging wall beam is thin or the green bar is exposed there can be ground control problems which are exacerbated by the stress fractured hanging wall. As a result welded steel mesh and split-sets are installed.

The in-stope welded mesh is installed conventionally in all gullies and raises, as well as in face areas where geological conditions dictate. The mesh in the gullies is normally installed to within 2 m of the gully face so it is not subjected to a lot of blast damage; however in the raises due to ledging the mesh is blasted onto. Therefore, blast-on behaviour could be observed.

3.2.6. Two Rivers Platinum (ARM-Impala JV)

Two Rivers is situated on the southern part of the eastern limb of the Bushveld complex. The geological sequence present comprises the Upper Critical Zone and the lower part of the Main Zone of the Bushveld Complex; as a result two economically viable reefs are mined for PGEs and these are the Merensky and UG 2 reefs. At the time of the observations, the mine was only exploiting the UG 2 reef.

Several geological and geotechnical domains are encountered underground. From the exposures and experience gained at the mine, five elementary Ground Control Districts (GCDs) have been identified on the UG 2 and Merensky (although not being mined) reef horizons. One of the GCDs was identified and called the Pothole GCD. In the vicinity of potholes observations suggest that the dip of the reef becomes somewhat steeper as it is dragged by the pothole. Chromitite stringers, which occur above the reef, have also been observed to cut down towards the reef, resulting in a thinner hanging wall beam closer to the pothole. The probability and frequency of low angled features are higher in pothole areas as well. Furthermore, the contact between the hanging wall pyroxenite (HW1) and anorthosite (HW2) appears to be dragged down towards the reef and anorthosite is known to be more brittle than pyroxenite.

These pothole characteristics result in difficulties with managing the ground and the primary support is not sufficient to control the ground. As a result, secondary areal coverage in the form of TSL is applied.

In-stope TSL is applied using pneumatic hand-held methods where geotechnical conditions dictate. The TSL is applied lagging behind the face as required; therefore observation of blast-on behaviour of the TSL is seldom possible. Observations were carried out in a production bord in a UG 2 horizon where TSL application was undertaken. Two TSL-based standards are applied according to ground conditions, viz with cables and without cables.

3.2.7. Tumela Mine, Anglo Platinum

The mining operation is located on the north-western sector of the Bushveld Complex. The two main reef bodies mined are the Merensky and the UG 2 Reefs. Observations were carried out in a UG 2 working area. The immediate hanging wall of the UG 2 is an altered olivine-rich poikilitic pyroxenite (harzburgite) layer which occurs at a maximum depth of 70 cm in the hanging wall. The altered nature of the pyroxenite results in time-dependant scaling of the hanging wall and the separation of 2 cm - 15 cm thick sheets of pyroxenite. These layers are relatively cohesion-less and warrant beam building through the use of bolts.

The UG 2 reef package often drops below its normal plane and this is referred to as ‘reef slumping’. The hanging wall beam is then disrupted by the mining not being able to closely follow the reef top contact, resulting in higher risks of exposing the harzburgite and experiencing FOGs. The site where observations were carried out was a slumped UG 2 reef horizon centre raise development access. Thin-strip welded straps (commercial name OSRO-straps) are typically used as areal coverage in the centre raise hanging-walls on the operation; however, the straps are susceptible to machinery damage during cleaning as well as blast damage during ledging.

As a consequence in-stope Carbontech V-Seal TSL was applied on a trial basis to the centre raise hanging wall in a selected stope using pneumatic hand-held methods to manage local geotechnical conditions. The TSL was applied right up to the ledging-face prior to ledging; as a result, blast-on behaviour of the TSL was observed. Information on the insitu age of the TSL was not directly available.

However, a 220 m length raise line had TSL applied at a rate of 20 m liner distance per day. The TSL was at least 11 days old prior to ledging. Investigations suggested that the TSL performed better than OSRO straps that had previously been used to manage reef slumping.

3.2.8. Lonmin plc Karee 4 Belt Lonmin

Karee 4 Belt is situated in the western limb of the Bushveld Complex and is approximately 30 km east of Rustenburg. The mine is currently exploiting the UG2 reef. The immediate hanging wall of the UG2 is a 40 cm thick gangue beam that resulted from the presence of chromitite stringers in the overlying pyroxenite. The mining standard is to mine out the hanging wall beam since it poses a risk of FOG if it is undercut. However; extracting the gangue beam together with the ore results in dilution and reduced ore grades. In an attempt to minimise ore dilution, the beam was undercut and the risk of FOG was managed by the installation of ultra-strength chain link (diamond) mesh with mechanical end tendons.

The mesh was installed during 2015 as a trial in a selected UG2 stope using conventional methods to manage local geotechnical conditions. The installation was done right up to the face; as a result, blast-on behaviour of the mesh was observed.

3.2.9. Dishaba Mine Anglo Platinum

Dishaba Mine is situated in the north-western sector of the Bushveld Complex in close proximity to Tumela mine and both mines are part of the Amandelbult mine complex. The main geological characteristics of the two mines are similar, however the ground control problems are Dishaba are compounded by intense jointing. The hanging wall is highly jointed by two prominent sets as well as randomly orientated joint sets. These joint sets easily form wedges when they interact with an altered olivine-rich poikilitic pyroxenite (harzburgite) layer which occurs at a maximum depth of 70 cm in the hanging wall of the UG2.

As a consequence the primary support system at Dishaba consists of 3 m cable anchors. When the jointing is intense, steel welded mesh is installed up to the stoping face. This allowed for the observation of blast on-performance of the mesh.

3.3. Observation checklist

Prior to the commencement of the underground visits a checklist that captures the site observations and data was developed. The information gathered in the checklist was under the following sub-sections:

- *Mining site:*
- *Mining practice*
- *Support standard*
- *Support installation*
- *Geotechnical characterisation*

3.3.1. Mining Site

The data gathered under this sub-section describes the mining site. It informs the reader of the mining company, the operation where the observations were carried out and the commodity mined for purposes of orientation. The information gathered then focuses onto a specific working place.

The mining environment that governs the type support systems which can be used is recorded here. Information such as the mining method, dip of the workings, mining height and length of the workings, all of which control the practicality of implementing a certain support system as described later in the research, are also described here.

3.3.2. Mining Practice

The objective of mining is to produce ore profitably and safely. So this sub-section looks at the mine's production expressed as m^2 mined per month and mining cycle in general. Under the mining cycle particular attention was paid to the support cycle and the support systems used. The impact of using permanent areal support at the expense of production is established through the determination of the actual mine production compared to the targets.

The support crew complement is recorded and is used to determine the installed cost of the support system which is a function of both material and labour cost. Factors that affect the quality of the installation such as the installation methods, drilling and blasting methods and quality assurance procedures are also recorded under mining practice.

3.3.3. Support Standards

The support standard for a particular working area was supplied by the operation. The underground site visits did not seek to validate the correctness of the standards but what was however recorded was the deviation from the standards. The deviation from the support standard meant that the quality of the support system was compromised. The compromised support installation will be incorporated in the ranking tool that is developed later in the research

3.3.4. Support installation

Ground control strategies may be provided by single support elements or a combination of support elements. Some underground environments may require more than one strategy; therefore more than one support element may be used. The individual support elements are integrated into a support system. The capability and performance of support systems is determined by the characteristics of the comprising elements.

The support installation looked at the type of the support installed together with its specifications. The specifications were obtained from the manufacturer, and it is important to note that each support element characteristic differs from the other e.g. bolts cannot be compared to mesh. The most important characteristics for areal support are discussed in the Chapter 4.

The observational area was demarcated particularly in the face area and any damage of the support elements was quantified. The probable reasons for the damage were established. These reasons were then used as part of determining the robustness of the support system as discussed later in the research.

The resources and equipment required to install the support as well the installation procedure are recorded here. Time studies to determine the installation cycle and the direct installation cycle are recorded here. The compliance of the installation to the support standard is also recorded.

3.3.5. Geotechnical Characterisation

Rock mass conditions and the geotechnical characteristics dictate if support should be installed. The following geotechnical aspects were observed and recorded during the site visits:

- *Understanding the hanging wall geology; rock strengths of the immediate hanging wall lithologies; presence of weak parting planes and average heights of partings*
- *The water and weathering conditions reduce the strengths of rock masses. The Geological Strength Index (GSI) of the exposed hanging wall was recorded since it gives an indication of the rock conditions.*
- *Some rocks are generally strong; however stress conditions can result in rock fracturing and disintegration. The underground inherent stresses were estimated and the observed stress effects were also recorded; and*
- *Any joints that were observed were mapped and characteristics noted i.e.; the dip, dip direction, spacing, alteration and roughness were recorded. Subsequent rock mass ratings such as the Rock Mass Rating (RMR) were deduced where required. Empirical methods were then used to give an indication of whether areal support is required.*

4. METHODOLOGY FOR SELECTING AREAL SUPPORT IN VARYING UNDERGROUND MINING ENVIRONMENTS

The development of the methodology for selecting permanent areal support was based on detailed observations and assessment of permanent areal support installations at several operations, Table 3-1, as well as South African and international literature.

Systematically, though with some subjectivity, the methodology relates the following aspects during decision making:

- *Mining environment;*
- *The capacity of the areal support;*
- *Performance factors (robustness of the support to endure rigors during installation and mining);*
- *Practicality; and*
- *Installed cost*.

An overview of the methodology is illustrated simplistically in Figure 4-1.

A ranking tool, based on this methodology, has been developed to enable the selection of areal support. The application of this tool is described in Chapter 5.

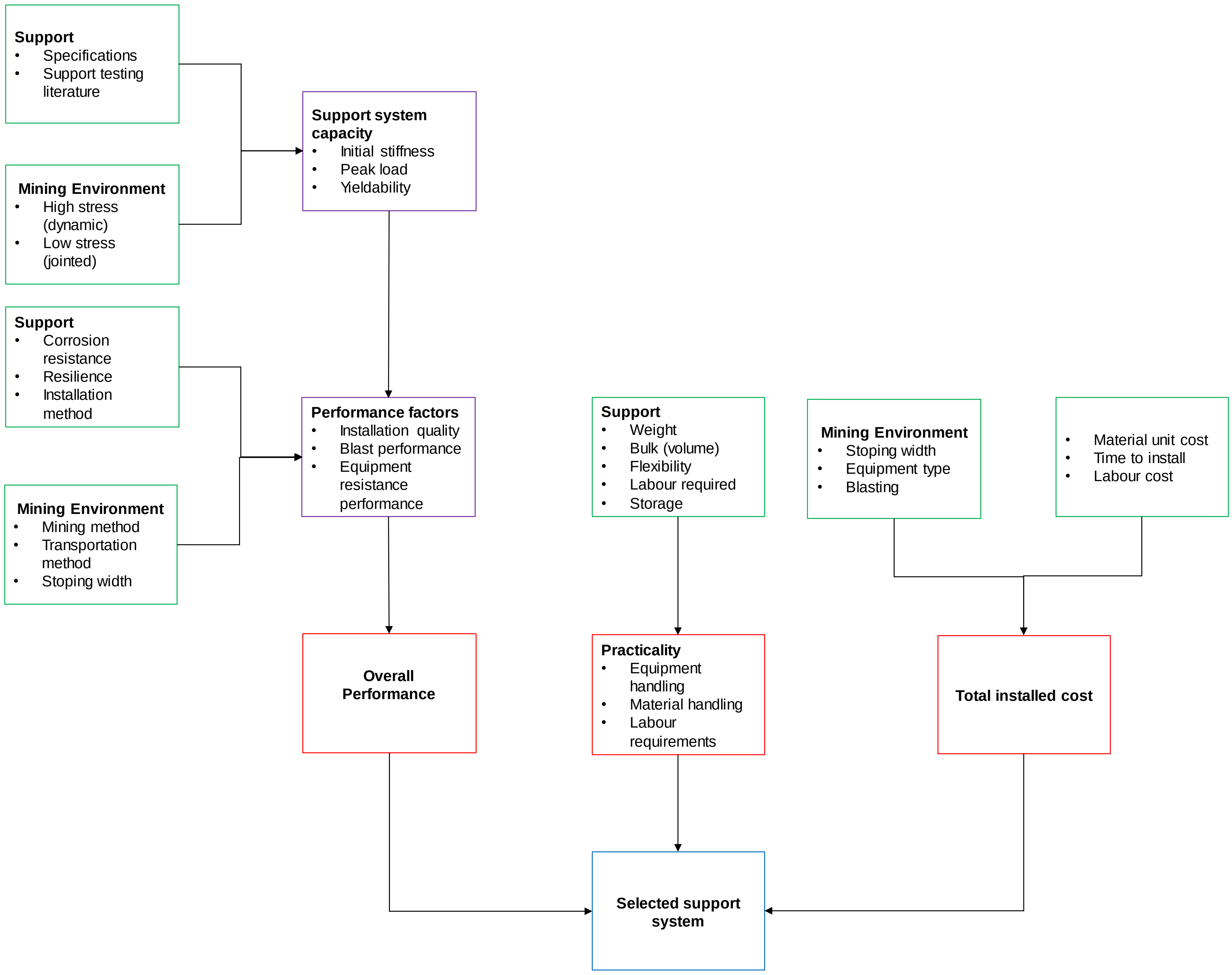

Figure 4-1: Overview of the methodology for the selection of permanent areal support

4.1. Mining environment

The stability requirements of the mining environment and the exposure of personnel within the environment inform the function that the support system is expected to provide. Aspects of the mining environment considered during support selection are differentiated in Table 4-1. An overview of the permanent areal support systems which are typical of the mining conditions (and combinations of these mining conditions) addressed in the project, are summarised in Appendix A.

Table 4-1: Aspects of the mining environment

Aspect	Variation
Mining depth (depth below surface, dbs)	Deep (≥ 1 500 m dbs); Intermediate (500 m ≤ dbs ≤ 1 500 m) and Shallow (≤ 500 m).
Stress	High stress vs low stress (in order to consider the susceptibility to- and the effects of- seismic activity, rock bursting, rapid deterioration of rock walls - as observed through fracturing, slabbing and deformation)
Structure (rock mass conditions)	Jointed vs unjointed rock masses (kinematic-driven wedge failures vs beam (shear) or stress-driven failures)
Location in working area	Mining face vs back area
Safety (through exposure of personnel)	Number of workers, duration of exposure and intensity of manual labour
Extent of mechanisation	Conventional vs semi-mechanised (bord and pillar) mining, manual vs mechanised
Stoping width	Low stoping widths vs high stoping widths
Corrosiveness	Dry vs water conditions, low vs high PH, low vs high sulphate content
Temparature	Cool vs hot working conditions
Humidity	Damp vs humid conditionss

4.2. Support capacity

Jager & Ryder (1999) describe the characteristics of support elements. The main characteristics are initial stiffness, peak load, yield, and energy absorption, which are illustrated in Figure 4-2. The capacity of areal support systems can be compared by considering these characteristics.

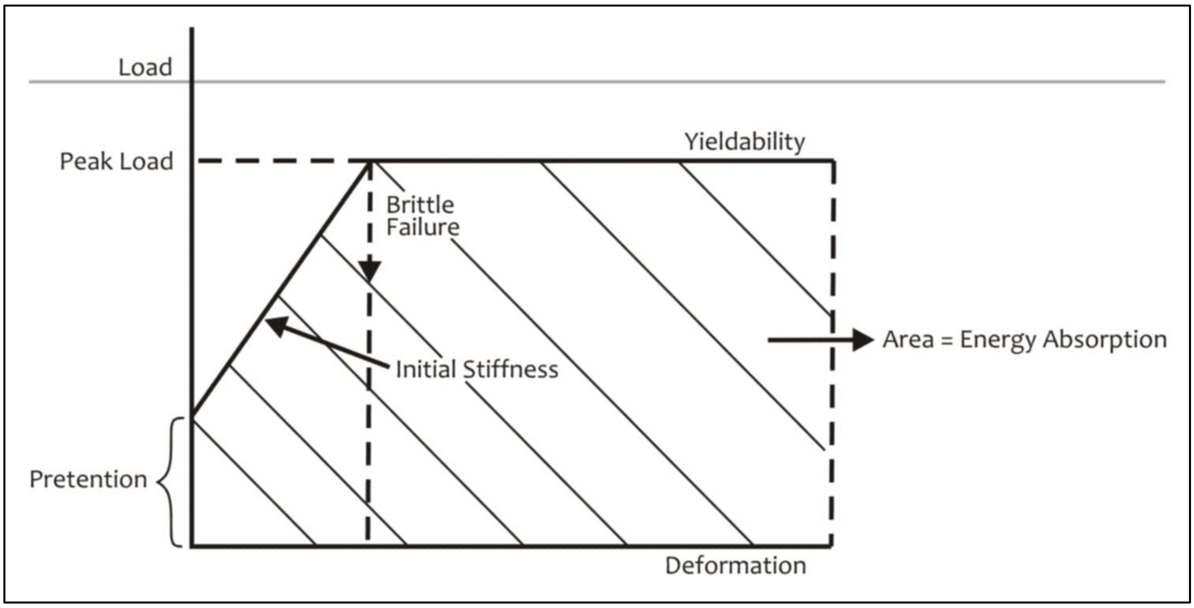

Figure 4-2: Key support characteristics (Jager & Ryder , 1999).

The initial stiffness is the rate at which load is developed within the support system with deformation of the rock mass. High initial stiffness is generally desirable as it prevents the unravelling of the rock mass. Shotcrete is an example of a support element with high initial stiffness whereas mesh is a low initial stiffness system, except its inherent stiffness (i.e. the stiffness of steel).

The peak load capacity is the maximum amount of stress that a support element can resist before it yields.

The total amount of deformation a support element undergoes beyond the peak load capacity prior to total failure is termed yieldability. The energy absorption capacity of a support system can be calculated from the area under a load deformation curve. In high stress or dynamic environments high yield and energy absorption capacity are required.

At present, there is no single source of comparable tests for the range of areal support systems available. The loading and boundary conditions in the various test programmes conducted for research purposes, differ considerably and therefore it is not possible to compare the support characteristics directly. Therefore, it is important to consider the support specifications and test results from a variety of research programmes in order to compare the different types of support.

4.2.1 Specifications

Support capacities of mesh and liners cannot be directly compared, therefore, the specifications in the ranking tool have been split into two segments namely mesh strand specifications and liner specifications. The product specifications of the support elements have been sourced from the suppliers.

Mesh strand specifications

The mesh specifications include the aperture size (which determines the number of wires per unit area); gauge (diameter) of the mesh strands and the strand tensile strength of the mesh. Using the stress (strength) and area relationship, the single strand strength which is strand tensile strength per unit strand area can be determined. Mesh characteristics are described in Chapter 2.4.

Liner specifications

TSLs and shotcrete were grouped as liners. Basic specifications are applied thickness, compressive strength and shear strengths. For shotcrete the peak load capacity can be determined by using the methodology developed in the SIM040204 research project (Joughin et. al. 2012), which is summarised in Chapter 2.2. TSL material characteristics are described in Chapter 2.3.

4.2.2 Mesh characteristics from literature

Mesh characteristics can also be compared using data from the Western Australia School of Mines (WASM) - Player et al. (2008) and the Canadian Handbook for Rockbursts (CHR) - Kaiser et al. (1996). The WASM tests are described in Chapter 2.4.2.

4.2.3. Shotcrete and TSL index tests

Yilmaz (2011) developed two new laboratory testing methods for the determination of TSL mechanical properties. The new testing methods determine the shear-bond between the rock substrate and TSL and the material shear strengths of TSLs. The study adopted two other testing methodologies with modifications and these are material tensile strength testing and tensile-bond strength testing. An overview of Yilmaz's (2011) testing methods has been discussed in Chapter 2.3.

4.2.4. Energy absorption and yield testing for mesh and shotcrete

Ortlepp & Stacey (1997) (see Chapter 2.5) conducted large scale tests on numerous areal support systems, including shotcrete, mesh and combinations with lacing and determined the maximum deflections and energy absorption capacity.

Potvin et al. (2010) attempted to collate energy absorption and deformation results from several research programmes (Kaiser et. al. 1996, Ortlepp & Stacey ,1997 and Player et al., 2008). It was necessary to assume corrections for the boundary conditions. The comparative results are presented in Figure 4-3. This graph is useful for comparative purposes.

4.3. Performance factors

Once installed underground, the support system may be subjected to mechanical actions of mobile machinery, blast effects and corrosion. These factors, together with the quality of the installation, may compromise the capacity of the support systems, which in turn negatively affects the performance thereof.

Connection points or overlaps between adjacent mesh panels are often the mesh system's weakest points. Adherence to the set out standard operating procedures (SOP) are emphasised. A deviation from the SOPs results in poor quality installation.

The mining environment can affect the performance. In narrow stopes, machinery damage is more likely. Corrosive water (low PH, high sulfate content) will be more aggressive. The type of explosives, blasting pattern and proximity of installed support to the mining face will increase the likelihood of blast damage.

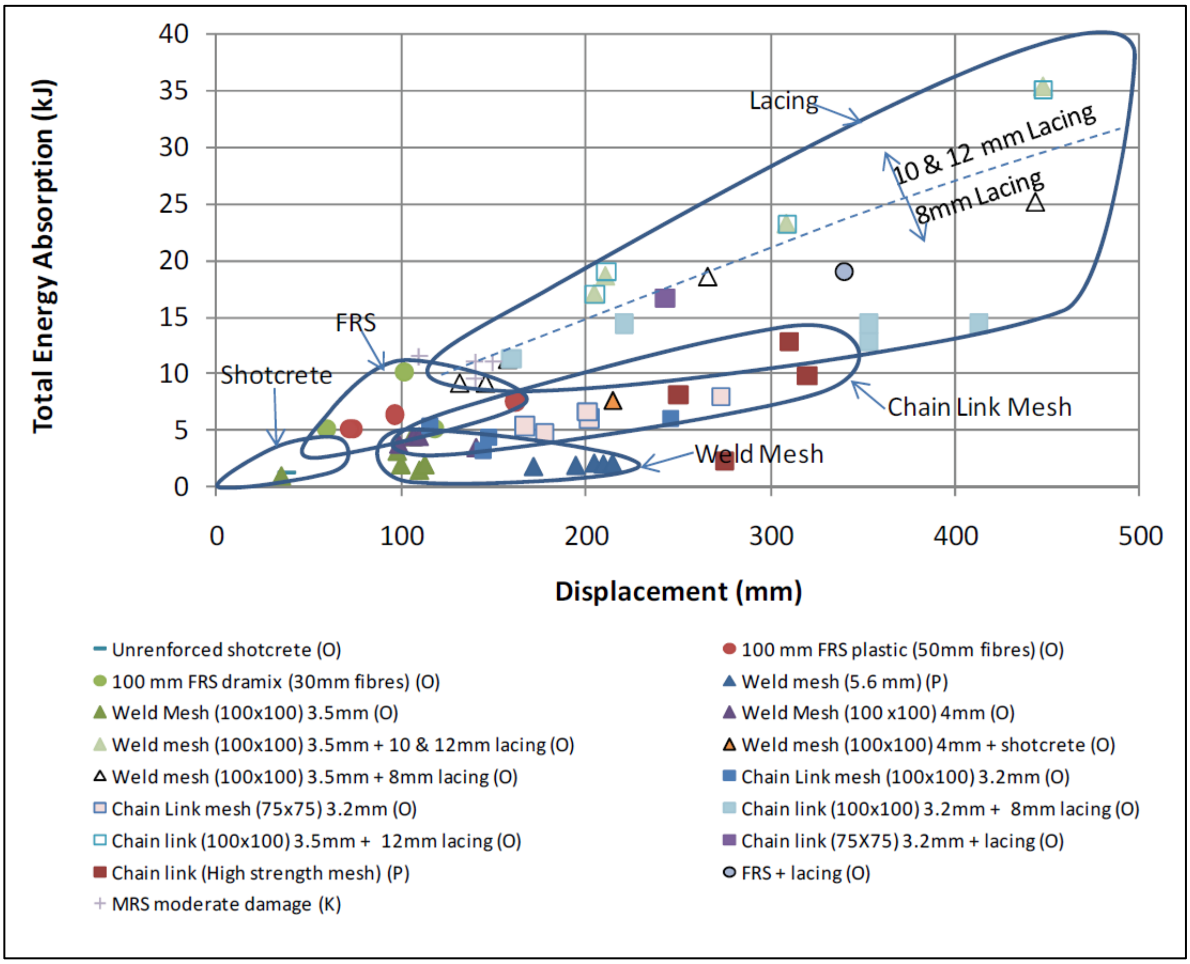

Figure 4-3: Compilation of drop tests performed on various surface support elements and reported by the following authors: Kaiser et al. (1996) (K); Ortlepp and Stacey (1997) (O); Player et al. (2008b) (P) after (Potvin et al., 2010)

4.4. Practicality

In addition to the performance of the support system, the practicality of installation of the system is also relevant in informing the selection of areal support. When assessing the practicality of the support installation, the following concepts are considered:

- Materials handling: Addresses the difficulty or ease of transporting the materials (support elements/units) to the working faces. The weight, volume (bulk) and flexibility of the elements dictate the difficulty or ease of handling. The storage of the support elements needs to be equally considered.

- Equipment handling: Assesses the difficulty or the ease of transporting equipment used in the installation of the support system.
- Overall handling: The concept which has a lower rating (i.e. materials handling or equipment handling) is considered as the overall handling rating. In other words, the worst-case handling determines the inefficiency of the installation of the system.
- Labour requirements: Assesses the number of people and level of mechanisation required in the installation of the support system.
- Installation: Assesses the difficulty or the ease of installing the support.

The practicality of the installation of the support system was found to be affected by the mining methods and/or stoping height or mining environment as well as the level of mechanisation.

4.5. Installed cost

The appeal of the support system may rate highly, but the financial costs may be prohibitive, and therefore are considered during the decision-making process. The total installed financial costs are derived using the installation time, the number of people involved in the installation, the labour costs as well as the material costs.

The decision as to whether the cost may be prohibitive depends on factors such as economics of the respective operations. Such factors may be highly confidential, highly variable or highly contextual.

5. RANKING TOOL FOR THE SELECTION OF AREAL SUPPORT IN DIFFERENT ENVIRONMENTS

A ranking tool for the selection of areal support, which is based on the methodology described in Chapter 4, is presented in Table 5-1. This represents data captured during underground visits and observations. The ranking tool comprises four broad analysis elements, namely support capacity, support performance factors, practicality and costs. It is important to note that the mining environment (section 4.1) plays a significant role in the allocation of ratings.

5.1. Support capacity rating

The support specifications and data from support testing (section 4.2) are presented with green shading in the ranking tool Table 5-1. Where the ranking tool support units are dissimilar to those tested, several corrections and/or assumptions have been applied (which are detailed in Appendix B) and presented with blue shading.

Initial stiffness, peak load and yield capacity ratings, ranging from 1 to 10, are then assigned to the support units on a comparative basis based on the support specifications, data and designer's judgement. A rating of 1 is undesirable whereas 10 is favourable. Table 5-2 summarises examples of how extreme key support characteristics are rated.

Table 5-1: Ranking tool for the selection of permanent areal support in underground mining environments

SUPPORT ASSESSMENT TOOL													
Analysis element	Mining Environment			High stress (dynamic) environment				Low stress and jointed (static) environment					
	Case Study			Case Study 1 Bambanani Gold Mine	Case Study 2 Bambanani Gold Mine	Case Study 5 Kloof 4 Shaft	Case Study 6 Tau Tona	Case Study 10 Dishaba	Case Study 8 Karee 4 Belt	Case Study 3 Booysendal Platinum - normal	Case Study 4 Booysendal Platinum - high	Case Study 7 Two Rivers	Case Study 9 Tumela
	Support system			Steel rope netting and hydrabolts	Steel rope netting	Weld mesh and hydrabolts	Weld mesh and split - sets	Weld mesh and cable anchors	Ultra strength chain link mesh and mechanical end anchors	Shotcrete and resin rebars	Shotcrete and resin rebars	TSL and resin rebars	TSL and mechanical end anchors
	Location			Stope	Stope	Gully	Gully	Gully	Stope	Bord	Access decline	Bord	Gully
	Stoping width (m)			2.2	1.5	1.8	2.3	2.5	1.5	2.4	3.8	2.3	1.6
	Depth of mining (m)			1950	1950	3 550	3 090	1 050	260	200	30	520	740
Support capacity characteristics	Specifications	Mesh strand specifications	Strand tensile steel strength (MPa)	900	900	400 - 600	400 - 600	400 - 600	1 770				
			Strand thickness (mm)	5.0	5.0	4.0	5.6	4.0	3.0				
			Stand strength (kN)	17.7	17.7	5.0 - 7.5	9.9 - 14.8	5.0 - 7.5	12.5				
			Apperture size (mm)	150 × 150	150 × 150	100 × 100	100 × 100	100 × 100	80.0				
		Liner Specifications	Liner thickness (mm)							50	50	8.0	8.0
			Compressive strength (MPa)							20 - 28	20 - 28	26.4	
	Comparative test results from literature	CHR	Peak load (kN)			12 -18	34 - 42	12 -18	64 - 76				
			Peak deformation (mm)			100 -150	150 - 200	100 -150	> 400 - 450				
			Stiffness (kN/m)			120	217	120	< 120				
			Ultimate deformation (mm)			125 - 175	175 - 250	125 - 175	> 400 - 450				
			Peak energy (kJ)			0.5 - 1.0	2.5 - 4.0	0.5 - 1.0	4.5 - 6.0				
			Total energy (kJ)			1.0 - 4.0	6.0 - 9.0	1.0 - 4.0	15 - 18				
			Design energy (kJ)			1.0	5.0	1.0	7.0				
		WASM	Peak load (kN)			43 -130	85 - 255	43 -130	45 - 145				
			Peak deformation (mm)			130 - 165	170 - 220	130 - 165	175 - 220				
			Ultimate deformation (mm)			100	220	100	220				
			Initial deformation (mm)			> 130	130.0	> 130	> 200				
			Total energy (kJ)			0.8 - 1	2 - 2.5	0.8 - 1	1.0 - 5.1				
		Index testing (Yilmaz, 2011)	Tensile strength (MPa)							0.5	0.5	<7.5	7.5
			Tensile-bond strength (MPa)							0.3	0.3	<2.5	2.5
			Shear-bond strength (MPa)							1	1	<6.5	6.5
			Material shear strength (MPa)							1	1	<6.5	17
		SIM040201 (Joughin et 2012)	Peak load (kN)							25	25		
		(Ortlepp and Stacey, 1997)	Total energy (kJ)			99 - 141	145 - 207	99 - 141	> 270	2 - 38	2 - 38		
			Deformation (mm)			3.5 - 4.5	10.0 - 13.0	3.5 - 4.5	15 - 26	0.8 - 1.3	0.8 - 1.3		
	Initial stiffness			3	1	6	7	6	7	9	9	8	8
	Yield capacity			8	8	4	6	4	8	1	1	2	2
	Load capacity			9	9	5	7	5	8	5	5	5	5
	Support system capacity rating			**7**	**6**	**5**	**7**	**5**	**8**	**6**	**6**	**6**	**6**
Performance factors		Installation Quality		8	6	8	9	5	9	9	9	8	8
		Blast performance		9	9	6	7	6	9	9	9	9	9
		Equipment damage performance		8	8	5	5	3	6	9	9	8	8
		Corrosion performance		8	8	9	9	5	10	10	10	10	10
		Performance rating		8	6	5	5	3	6	9	9	8	8
		Performance factor		0.8	0.6	0.5	0.5	0.3	0.6	0.9	0.9	0.8	0.8
		Overall performance rating		**5**	**4**	**2**	**3**	**2**	**5**	**6**	**6**	**5**	**5**
Practicality		Equipment handling		9	10	9	9	9	9	8	8	8	5
		Materials handling		7	8	5	4	5	8	6	5	6	4
		Overall handling		7	8	5	4	5	8	6	5	6	4
		Labour requirements		2.5	2.5	2.5	2.5	2.5	5	8	7	8	8
		Installation		6	8	6	5	6	7	8	8	8	4
		Overall practicality rating		**5**	**6**	**4**	**4**	**4**	**7**	**8**	**7**	**8**	**5**
Overall support system rating				**5**	**5**	**3**	**3**	**2**	**5**	**6**	**6**	**6**	**5**
Cost		Installation time (m^2/ hr)		30	30	11	18	21	21	18	23.6	19	41
		Material cost (R/m^2)		196.76	155.00	116.39	240.62	200.42	220.19	432.56	432.56	389.78	406.57
		Labour cost (R/m2)		14.38	14.38	22.88	22.37	8.09	8.65	21.09	24.13	16.65	4.93
		Total installed cost (R/m^2)		211.14	169.38	139.27	262.99	208.51	228.84	453.65	456.69	406.43	411.50

Legend	
	Not applicable
	Test result
	Adjusted results
	Specifications
	Rating

Table 5-2: Rating key support characteristics

Characteristic	Support Unit/System	Rating
Initial Stiffness	Steel rope nets supposed by packs and no bolts	1
	Bolted shotcrete	10
Peak Load	3.15 mm diameter mesh (Steel strength 400 MPa)	1
	5 mm diameter steel rope mesh (Steel strength 900 MPa)	10
Yield Capacity	Unreinforced shotcrete	1
	Ultra-strength chain-link mesh	10

There has not been large-scale laboratory testing of TSLs yet. Ratings and comparisons of TSLs' key support characteristics to other support units are based on the TSLs' mechanical properties and in-situ observed performance. TSLs are expected to have a higher initial stiffness when compared to mesh but more flexible than shotcrete. So, the initial stiffness of TSL is expected to be greater than that of mesh and less than that of shotcrete.

TSL is generally expected to withstand larger deformations (Tannant, 2001) when compared to shotcrete which is known to fail over small deformations (Ortlepp & Stacey, 1997). On the other hand, TSL when compared to mesh fails at much smaller deformations. Therefore, the yield capacity of TSL is expected to be greater than that of shotcrete but less than that of mesh. TSL performance characteristics ratings heavily depend on the judgement and experience of the designer. A firm understanding of the mechanical properties of the TSL and the mining environment in which it is applied becomes paramount.

The requirements of the support system to be installed depend on the mining environment. Underground observations were carried out in two contrasting mining environments, namely high stress (dynamic) environment and a low stress jointed (static) environment. The suggested weighting factors for these mining environments are shown in Table 5-3. Weighting factors for an environment should add up to one. Initial stiffness is considered important in both environments, to avoid unravelling of the rock mass. Yield capacity is important in a high stress and dynamic environment, but not in a low stress (jointed) environment. These weighting factors are simply a guide.

Table 5-3: Suggested weighting factors for mining environments

		High stress and dynamic	Low stress (jointed) environment
Support system capacity	**Initial stiffness**	0.3	0.3
	Yield capacity	0.4	0
	Load capacity	0.3	0.7

The support system capacity is thus calculated using the equation:

$$SSC_{rating} = IS_{MEF} \times IS_{rating} + YC_{MEF} \times YC_{rating} + LC_{MEF} \times LC_{rating} \qquad \text{(Equation 23.)}$$

Where

IS_{MEF} = Initial stiffness mining environment factor

IS_{rating} = Initial stiffness rating

YC_{MEF} = Yield capacity mining environment factor

YC_{rating} = Yield capacity rating

LC_{MEF} = Load capacity mining environment factor

LC_{rating} = Load capacity rating

5.2. Performance rating

When one of the performance parameters has been compromised the performance of the whole system is compromised. Therefore, the performance of the support system is equivalent to the minimum of the support system performance parameters. The equation for the performance rating is:

$$Per_{rating} = \min(IQ_{rating}; BR_{rating}; ER_{rating}; CR_{rating}) \qquad \text{(Equation 24.)}$$

Where

IQ_{rating} = Installation quality rating

BR_{rating} = Blast performance rating

ER_{rating} = Equipment performance rating

CR_{rating} = Corrosion performance rating

The overall performance rating of the support system is a function of the system capacity and its performance rating expressed as a performance factor (PF). Guidelines on the performance ratings are given in Table 5-4. The performance factor is decile expression of the performance rating, $PF = \frac{Per_{rating}}{10}$. The overall system performance is calculated using the equation:

$$OP_{rating} = PF \times SSC_{rating} \quad \text{(Equation 25.)}$$

5.3. Practicality rating

The overall practicality is a weighted rating of the overall handling, labour requirement and installation. Guidelines and descriptions of practicality rating are tabulated in Table 5-5. The overall practicality rating is calculated using the equation:

$$P_{rating} = w_{OH} \times OH_{rating} + w_{LR} \times LR_{rating} + w_I \times I_{rating} \quad \text{(Equation 26.)}$$

Where

P_{rating} = Practicality rating

W_{OH} = Overall weighting factor

OH_{rating} = Overall handling rating

W_{LR} = Labour requirements weighting factor

LR_{rating} = Labour requirements rating

W_I = Installation weighting factor

I_{rating} = Installation rating

5.4. Overall support system rating

The overall support system rating is a weighted sum of the overall performance rating and the overall practicality rating, and is expressed using the equation:

$$OSS_{rating} = w_{op} \times OP_{rating} + w_p \times P_{rating} \quad \text{(Equation 27.)}$$

Where

OSS_{rating} = Overall support system rating

OP_{rating} = Overall performance rating

w_{op} = Overall performance weighting factor

P_{rating} = Practicality rating

w_p = Practicality weighting factor

5.5. Installed Cost

The following calculation procedure was used in the determination of the total installed cost:

$$Total\ Installed\ cost\ (R/m^2) = Material\ cost\ (R/m^2) + \ Labour\ Cost\ (R/m^2)$$

Where

Material cost is calculated based on the quantities of support units installed per m^2

Labour cost is determined as follows:

$$Labour\ cost\ \ (R/m^2) = \frac{Labour\ cost\ \ R/hr}{Installation\ rate\ \ m^2/hr}$$

$$Labour\ cost\ \ R/hr = \frac{cost\ per\ person\ (R/month)\ \ \times people\ involved}{hours\ worked\ per\ month\ \ (hr/month)}$$

Table 5-4: Description and ratings for 'effectiveness of support'

Class	Description of classification	Rating
Blast performance		
A	Heavily damaged cannot be fixed, new support to be installed	0 - 2.5
B	Moderate damage, fixed by rehabilitation	2.5 - 7.5
C	Little damage that does not need rehabilitation	7.5 - 10
D	No damage at all	10
Equipment performance		
A	Heavily damaged cannot be fixed, new support to be installed	0 - 2.5
B	Moderate damage, fixed by rehabilitation	2.5 - 7.5
C	Little damage that does not need rehabilitation	7.5 - 10
D	No damage at all	10
Corrosive performance		
A	Completely corroded	0 - 2.5
B	Severe damage	2.5 - 7.5
C	Superficial rusting	7.5 - 10
D	No visible rusting	10

Table 5-5: Descriptions and ratings for 'practicality'

Class	Description of classification	Rating
	Labour requirements	
A	Highly labour intensive and manual installation	0 - 2.5
B	High labour intensity and semi -mechanised installation	2.5 - 5
C	Low labour intensity and semi - mechanised installation	5 - 7.5
D	Low labour intensity, highly mechanised installation	7.5 - 10
	Installation	
A	Extremely difficult to apply or install	0 - 2.5
B	Very difficult to apply or install	2.5 - 5
C	Fairly easy to apply or install	5 - 7.5
D	Easy to apply or install	7.5 - 10
	Materials handling	
A	Very difficult to transport to the working face	0 - 2.5
B	Difficult to transport to working face	2.5 - 5
C	Fairly difficult to transport to the working face	5 - 7.5
D	Easy to transport to the working face	7.5 - 10
	Equipment handling	
A	Very difficult to transport to the working face	0 - 2.5
B	Difficult to transport to working face	2.5 - 5
C	Fairly difficult to transport to the working face	5 - 7.5
D	Easy to transport to the working face	7.5 - 10

5.6. Application of ranking tool (A worked example: assessment)

The ranking tool presented in Table 5-1 was applied to all case studies (champion mines) described in Section 3.2. A detailed description of how the rating for all the support units was done is given in Appendix A1 to Appendix A10. For illustrative purposes on how the ratings for Case Study 1: Bambanani high stoping width using steel rope netting and Hdyrabolts is discussed under this sub-section.

5.6.1. Mining Environment:

The mining environment is described as deep level mining in high stress, seismically susceptible, tabular hard rock environment using conventional (manual) stoping methods in ≥ 1.5 m stoping height. Based on the mining environment and using Table

5-3, the initial stiffness, yield and peak capacities weighting factors towards the support system capacity will be 0.3, 0.4 and 0.3 respectively.

5.6.2. Support system capacity rating

Strand tensile strengths of 17.7 kN was calculated (based on the strand thickness and tensile steel tensile strength) for the steel nets and was the strongest amongst the observed mesh which included ultra-strength mesh produced from steel of 1.8 GPa. As a result, the steel netting was given a load capacity rating of 9.

The observed steel rope nets were not tautly installed against the stope back (hanging wall), and were a classic example of passive support thus compromising the initial stiffness of the mesh system. Two systems of steel rope nets were installed in the mine; one involved the use of timber packs to hold the mesh against the hanging resulting in loose areal coverage between the timber packs. Whereas in the other system the steel rope nets were held against the stope hangingwall by way of timber packs and tendons providing for a relatively taut but still considerably loose areal coverage.

Elsewhere, observations carried out on welded and ultra-strength chain-link meshes were generally stiffer and more tautly installed against the stope back as compared to the steel rope nets. This suggests higher initial stiffness of the weld and ultra-strength mesh systems.

An estimate of the initial stiffness of the mesh can be calculated from load – deformation curves as the slope of the curve before reaching the design load. The initial stiffness rating of the steel rope nets was based on an extrapolation of the weld and chain-link mesh laboratory test results and underground observations due to the absence of steel rope nets laboratory test results. The steel rope net was expected to have a lower initial stiffness than both welded mesh and chain link mesh. Therefore, an initial stiffness rating of 3 was given for this system.

Human, (2005) performed drop tests on the steel rope nets. The actual results obtained were contentious because the values obtained for the energy absorption were much

much higher than expected when compared to other mesh test results. However, it was not disputed that steel welded mesh has very high energy absorption capacities based on the peak load capacity of the mesh as well as its ability to deform. Consequently, the yield capacity for the steel rope netting was rated as 8. A Support System Capacity (SSC) rating was calculated to be 7 using Equation 23.

5.6.3. Performance rating

The use of tendons ensured that the steel rope netting was not fully sagging and that the steel rope net offered some initial stiffness. This generally meant that the quality of installation was good and a rating of 8 was given. During observations, there were no indications of blast and equipment damage on the netting and as a result, both equipment and blast damage performance factors were rated as 9. The steel rope net had superficial rusting, and this resulted in a corrosion rating of 8. These ratings for equipment and blast damage as well as corrosion resistance were guided by the descriptions in Table 5-4.

The performance rating was determined as equal to the lowest performance factors and was rated as an 8 using Equation 24. A performance factor of 0.8 was then calculated which is an input of the Overall Performance (OP) rating which can be determined using Equation 25. An OP rating of 5 was then consequently determined for this system.

5.6.4. Practicality rating

Descriptions in Table 5-5 were used to guide practicality ratings. The installation of steel rope netting is done manually by hand. However, when the netting is used together with tendons, airlegs need to be transported into the stopes to drill holes for the tendons. The transportation and manoeuvring of airlegs in the stopes is done routinely and is not a difficult task, therefore equipment handling was given a rating of 9.

The steel rope netting is flexible and can be easily rolled and transported into the stopes on the scraper winch rope. However, with the need to transport tendons the material handling becomes fairly difficult therefore a rating of 7 was used as described in Table

5-5. The installation of the steel rope nets is highly labour intensive and does not involve any mechanisation; therefore a rating of 2.5 was used. The installation of the steel rope net was relatively straightforward. However, the use of the hydrabolt tendons increased the difficulty of the areal support installation and the installation was rated 6. These ratings were used together with Equation 26 to determine the overall practicality rating of 5.

5.6.5. Overall support system rating

Mining was done in the shaft pillar where seismicity and subsequent hanging wall shakedowns are prevalent. Thus, the good performance of the support systems became of prime importance as compared to the practicality. As such, weighting factors of 0.7 and 0.3 were decided upon for the overall performance and overall practicality. An overall support system rating of 5 was calculated using Equation 27 which is a weighted sum of the overall performance rating and the overall practicality rating.

5.7. Application of ranking tool (worked example: design approach)

A conventional deep level gold mine with a stoping width of 1.5 m is planning on shaft pillar extraction at 2.3 km below surface. The hangingwall in the current workings is intensely stress fractured and comprises of interbedded lithologies thus posing a risk of FOGs. The risk of FOGs is exacerbated by frequent seismic events experienced at the mine.

There is consensus amongst the planning team, that a robust support system comprising friction tendons and areal support should be used in-stope. The rock engineer has been tasked to recommend the most appropriate areal support from a choice of steel welded mesh, ultra-strength chain link mesh and TSL. The specifications of the support units are given in Table 5-6

The design methodology proposed in this research is used to determine the most appropriate areal support unit. The selection method is explained in the sub-sections.

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5.7.1. Mining environment

The mining environment is described as high stress deep level, high stress, and seismically active environment. Therefore, using Table 5-3, the initial stiffness, yield and peak capacities weighting factors towards the support system capacity will be 0.3, 0.4 and 0.3 respectively.

5.7.2. Performance rating

Installation Quality

The use of tendons will ensure that both the weld and chain-link mesh will not be sagging, consequently offering good initial stiffness. This generally means that the quality of installation will be good and rated as 9. The quality of TSL application is affected by how clean the hangingwall is, as well as TSL thickness consistency. During underground visits, hangingwall cleaning was generally done well is not expected to be a problem however, TSL thickness consistency was deemed a lot difficult to determine. As a consequent a rating of 8 is appropriate for TSL quality of installation.

Blast performance

An 8 mm thick steel rope is woven to the chain-link mesh, the rope aids the chainlink mesh with energy absorption capabilities of fly rock during blasting. Blast-on behavior of the TSL and mesh was observed during site visits. Both TSL and chain-link mesh were observed to have minimum blast damage, with no rehabilitation required. According to descriptions given in Table 5-4 their blast performance are rated 9. Steel weld mesh was observed to have minor damages that could be fixed with some minor rehabilitation and is rated 7.

Equipment performance

When installed in the stopes, both weld and chain-link mesh can be easily damaged by equipment. The damage in both instances was moderate and required rehabilitation of the support units for optimal performance. Superficially, it was observed that damage on weld mesh was more than chain-link mesh. Therefore, using Table 5-4 the equipment performance for the weld and chain-link mesh can be rated as 5 and 6 respectively. From underground observations, TSL was subjected to equipment action, although there was damage it was minor and no rehabilitation was required. Therefore, an equipment performance rating of 9 is assigned for TSL.

The performance rating in Table 5-6 is carried out as described in Section 5.2.

5.7.3. Practicality rating

The practicality rating will be done as explained in Section 5.3. This sub-section that follow describe how the practicality component inputs in Table 5-6 were rated. Suggested ratings and descriptions for the practicality components are presented in Table 5-5.

Equipment handling

Both weld and chain-link mesh are installed by hand, with the only machines required being airlegs to install the tendons. Airlegs are routinely used in conventional underground mines and they are easy to transport therefore, a rating of 9 was assigned for both units. TSL is applied using a pneumatic spraying machine (which is feeder hoper, mixing drum, fitted with pneumatic pumps and pressure hoses) which weigh at least 60 kg. These machines are moved in the stopes is through confined gullies which are often full of broken ore. As per Table 5-5 a rating of 4 is therefore assigned for TSL equipment handling.

Materials handling

TSL is normally supplied in bags weighing 20 kg which makes its transportation extremely difficult. The 5.6 mm weld mesh is extremely stiff and has a relatively higher weight per unit area as compared to the lighter and more flexible chain-link mesh. Materials transport in a conventional stopes is through mono-winch ropes. Consequently, the transport of TSL into the stopes is very difficult, whilst it is difficult and easy for weld and chain-link mesh respectively. According to Table 5-5 rating of 2.5, 4 and 8 can be assigned for TSL, weld and chain-link mesh materials handling ratings respectively.

Installation

The installation of weld mesh and chain-link mesh can be described as fairly easy although it is slightly more difficult with weld mesh due to its stiffness and weight. Once the TSL machine has been set-up the application of TSL is easy. Using Table 5-5, the ease of installation for weld mesh, chain-link mesh and TSL can be rated as 5, 7 and 8 respectively.

Having determined the support system capacity, performance and practicality ratings as described above, the most appropriate areal support unit is then determined as described in Section 5.1 to 5.4 as shown in Table 5-6. Based on the results presented, the ultra-strength chain-link mesh is the most appropriate areal support unit to use..

Table 5-6: Stope support selection tool.

SUPPORT ASSESSMENT TOOL						
Analysis element	Mining Environment			High stress (dynamic) environment		
	Areal Support Unit			Weld mesh	Ultra strength chain link mesh	TSL
	Location			Stope		
	Stoping width (m)			1.5		
	Depth of mining (m)			2 300		
Support capacity characteristics	Specifications	Mesh strand specifications	Strand tensile steel strength (MPa)	400 - 600	1 770	
			Strand thickness (mm)	5.6	3.0	
			Stand strength (kN)	9.9 - 14.8	12.5	
			Apperture size (mm)	100 × 100	80.0	
		Liner Specifications	Liner thickness (mm)			8.0
			Compressive strength (MPa)			26.4
	Comparative test results from literature	CHR	Peak load (kN)	34 - 42	64 - 76	
			Peak deformation (mm)	150 - 200	> 400 - 450	
			Stiffness (kN/m)	217	< 120	
			Ultimate deformation (mm)	175 - 250	> 400 - 450	
			Peak energy (kJ)	2.5 - 4.0	4.5 - 6.0	
			Total energy (kJ)	6.0 - 9.0	15 - 18	
			Design energy (kJ)	5.0	7.0	
		WASM	Peak load (kN)	85 - 255	45 - 145	
			Peak deformation (mm)	170 - 220	175 - 220	
			Ultimate deformation (mm)	220	220	
			Initial deformation (mm)	130.0	> 200	
			Total energy (kJ)	2 - 2.5	1.0 - 5.1	
		Index testing (Yilmaz, 2011)	Tensile strength (MPa)			7.5
			Tensile-bond strength (MPa)			2.5
			Shear-bond strength (MPa)			6.5
			Material shear strength (MPa)			6.5
		(Ortlepp and Stacey, 1997)	Total energy (kJ)	145 - 207	> 270	
			Deformation (mm)	10.0 - 13.0	15 - 26	
	Initial stiffness			7	7	8
	Yield capacity			6	8	2
	Load capacity			7	8	5
	Support system capacity rating			**7**	**8**	**5**
Performance factors	Installation Quality			9	9	8
	Blast performance			7	9	9
	Equipment damage performance			5	6	8
	Corrosion performance			9	10	10
	Performance rating			5	6	8
	Performance factor			0.5	0.6	0.8
	Overall performance rating			**3**	**5**	**4**
Practicality	Equipment handling			9	9	4
	Materials handling			4	8	2.5
	Overall handling			4	8	2.5
	Labour requirements			2.5	5	8
	Installation			5	7	8
	Overall practicality rating			**4**	**7**	**5**
Overall support system rating				**3**	**5**	**4**

6. CONCLUSIONS

The methodology for the selection of areal support was based on detailed assessment of permanent areal support installations at several operations as well as South African and international literature. The design methodology culminated in a support assessment tool. The support assessment tool allows for a systematic, (though with some subjectivity) a methodology for the decision-making process in the selection of areal support.

The mining environment and the exposure of personnel within the environment informs the most important support specifications and characteristics, which in turn determine the Support System Capacity. The Support System Capacity rating is a weighted sum of the of initial stiffness, load and yield capacity ratings. The weighting factors used for the initial stiffness, load and yield capacities are all functions of the mining environment.

Support performance is not only dependent upon its capacity but is affected by other things that have been termed performance factors such as installation quality; blast resistance performance; resistance to equipment damage and corrosion resistance performance. The research argues that if any one of the performance factors is compromised, the whole system will be compromised. Consequently, the support system capacity rating is downgraded to take into account the effects of performance factors resulting in what has been termed the overall support performance. Tables that assist the designer with assigning corresponding ratings for the performance ratings were developed.

The support system can be technically sound and adequate from a performance point of view. However, it may not be practically installed underground. Consequently, the developed methodology makes provision for the overall practicality rating, which is a weighted sum of the overall handling (equipment and material); labour requirements and ease of installation ratings. Similar to performance factors, tables have been developed that guide in the assigning of values for each practicality component. The overall support system rating is a weighted sum of the overall performance rating and the overall practicality rating.

Recommendations

The design methodology developed assumes that the decision of installing areal support would have been reached; as such it weighs heavily on the side of the support characteristics. For further development of the design methodology it is recommended to incorporate geology as rock structures and rock mass quality in the ranking tool.

Initial stiffness, yield capacity and load capacity ratings in the ranking tool are a function of the support specifications and performance based on laboratory results. However, some support specifications, for example mesh and TSL specifications, cannot be compared directly, nor can laboratory results from different test rigs due to dissimilar boundary and testing conditions. For consistent quantitative analysis of the different support units, the study recommends testing the support units which are intended to be ranked under similar conditions.

Underground shotcrete design methodology follows a rigorous and mechanistic design process based on work done by Joughin, et al., (2012). The same cannot be said for TSL and mesh. There has been a dearth of mesh and TSL underground design methodologies both in literature and from data collected from champion mines. The designs are largely based on trial and error as well as experience. This often results in non-optimized support systems. The research therefore recommends detailed underground design methodologies for mesh and TSL.

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Appendix A

Overview of permanent areal support systems being currently implemented or trialled in South African tabular gold and platinum underground mines

Appendix A-1. Steel rope netting with tendons

Deep level mining in high stress, seismically susceptible, tabular hard rock environment using conventional (manual) stoping methods in ≥ 1.5 m stoping height

Overview

The support system is representative of design approaches universally encountered throughout the platinum and gold mining operations selected for the project.

A strand tensile strength of 17.7 kN was calculated for the steel nets and was the strongest amongst the observed mesh which included ultra-strength mesh produced from steel of 1.8 GPa. As a result the steel netting was given a load capacity rating of 9.

The observed steel rope nets were not tautly installed against the stope back, and were a classic example of passive support thus compromising the initial stiffness of the mesh system. Two systems of steel rope nets were installed; one involved the use of tendon support to hold the mesh against the hanging whereas in the other system the steel rope nets were held against the stope back by way of timber packs resulting in loose areal coverage between the timber packs.

The observed welded and ultra-strength chain-link meshes were generally stiffer and were more tautly installed against the stope back compared to the steel rope nets. This suggests higher initial stiffness of the weld and ultra-strength mesh systems. Observations were carried out on 4.0 mm and 5.6 mm diameter welded mesh, and it is intuitive that the larger diameter welded mesh was stiffer.

An estimate of the initial stiffness of the mesh can be calculated from load – deformation curves as the slope of the curve before reaching the design load. The initial stiffness rating of the steel rope nets was based on an extrapolation of the weld and chain-link mesh lab test results and underground observations due to the absence of steel rope nets laboratory test results. The steel rope net was expected to have a lower initial stiffness than both welded mesh and chain link mesh. Therefore an initial stiffness rating of 3 was given for the system.

The use of tendons ensured that the steel rope netting was not fully sagging and that the steel rope net offered some initial stiffness as stated. This generally meant that the quality of installation was good and a rating of 8 was given. During observations there were no indications of blast and equipment damage on the netting; as a result both equipment and blast damage performance factors were rated as 9. The steel rope net had superficial rusting and this resulted in a corrosion rating of 8. The overall performance rating was determined as equal to the lowest performance factors and was rated as an 8.

The installation of steel rope netting is done by hand. However, when used together with tendons the airlegs need to be transported into the stopes. The transportation and manoeuvring of airlegs in the stopes is not a difficult task therefore equipment handling was given a rating of 9. The steel rope netting is flexible and can be easily rolled and transported on the scraper rope. However, with the need to transport tendons, the requirement for material handling increases, therefore a rating of 7 was used. The installation of the steel rope nets is highly labour intensive and does not involve any mechanisation; therefore a rating of 2.5 was used. The installation of the steel rope net was relatively straightforward. However, the use of the Hydrabolt tendons increased the difficulty of the areal support installation and the installation was rated 6.

Design methodology

- Support performance: result obtained from testing steel cable safety nets through **drop testing** methods, measuring impact velocity and energy impulse, yield and displacement measurements and rupture load (Human, 2005), in conjunction with **issue based risk assessment**.
- Support material and performance specifications determined through the **empirical** evaluation of potential rockfalls, based on available **fall of ground** (FoG) records.
- Safety risk mitigation requirements for workers determined through discussion with **production personnel**.
- Performance monitoring measured in terms of mine-safety results which indicated positive effects.

- Quality assurance monitoring was carried out by means of selecting samples for drop testing in a workshop, where a 300 kg concrete weight is dropped onto a net secured with mechanical props. A sample is said to have passed if no more than one node is either displaced or disconnected (where a node is a crimped joint at each strand crossover point).
- Costing, while not specifically a support performance design criterion in terms of safe worker protection, it is nevertheless a necessary consideration in relation to the mine design objectives for profitability. Although there is a moderate cost associated with this support methodology (≈ R300 per m^2 for net, props and tendons), the cost of materials is offset against the benefits for safe production.
- Additional benefits to the system: handling and draping across the face for face burst protection.

Practical description of the system

Permanent steel rope netting is installed in-stope together with rapid yielding hydraulic props (RYHP) and tendons to provide permanent in-stope areal cover up to the face for the working team. Backfill is installed in the back area up to a specified distance from the face. Before each successive blast, the rope net is rolled back up to the closest line of RYHP's from the face and the face is blasted onto the net and the props. Very little (if any) observable blast damage of the nets, either in terms of broken nodes, warped strands or broken strands was observed. The good blast and equipment performances are reflected by blast and equipment performance ratings of 9 and 8 respectively.

Stoping was being carried out in a high stress, seismically susceptible, high stoping width (≥ 1.5 m), moderately dipping (30° inclination) horizon where a hanging-wall shale layer was mined out as part of the mining cut to expose an interbedded waxy brown quartzite. The structure of the quartzite is characterised by brittle stress fracturing which frequently results in fallout of the rock mass surrounding the tendon support. As a result, the occurrence of falls of ground as well as the risk of injury, due to limited areal coverage between the tendons, is mitigated through the use of steel rope netting (critical design consideration).

Benefits of the steel rope netting with tendons include:

- Excellent retention of rockfalls, both gravitational and seismic shakedown.
- Addresses limitations of tendon support to retain a brittle, stress-fractured hanging-wall.
- Complete removal of areal support prior to each blast is not required, the safety net is rolled to the last line of props, which saves time and reduces worker exposure (critical ergonomic / safety and economic design and selection criterion)
- Permanent protection for workers and limitation of falls of ground (logistics and production restrictions) in the back area and face area is provided resulting in improved safety performance for the mine workers and limited interruptions (therefore improvement) to productivity. The steel nets showed superficial rusting in the back areas and were assigned a corrosion performance rating of 8.
- Attachment of nets to hanging-wall at tendons and props.

Limitations of the steel rope netting include:

- Labour intensive.
- Strand thickness (gauge), aperture and net span (area) are directly related to the weight of the material and how unwieldy the net is to manoeuvre.
- Manual tensioning and fastening with "S" or similar type steel hooks. This results in sagging of the net (exacerbated in stopes without tendons for fastening) which allows rockfalls to gain momentum and excessively sag the net on impact. The net support is therefore a passive system. This affects the initial stiffness of the steel net which was consequently rated as 3.
- Fastening of contact points between successive nets using "S" or other type steel hooks is informally designed, which must be actively controlled as a quality assurance function during the installation of the areal support.
- "Nip" points between RYHP, net and hanging-wall that present potential damage points to the material and nip points for labour (injuries)
- Interference with equipment (scraper damage).
- Expensive materials. The steel rope netting system was considered feasible from an economic perspective only because of the low relative cost to yield ratio of the local orebody (low relative cost against high yield).

- Low level design considerations. Selection of the steel rope netting is largely driven by empirical (experiential) input, professional opinion and laboratory type drop testing to determine the net capacity for arresting falls of ground.

Using an empirical approach (viz. a combination of personal experience and professional judgement) as a basis for the selection of permanent areal support in a working face (or elsewhere) was common at all of the mining operations selected for the study and, based on the researcher's general experience, is a fair representation of current practice in the industry at large in South Africa. "Quantified design" as such, through analytical and integrative interpretation of statistically representative safety, rock mass, productivity, material cost and material performance specifications is very rarely carried out, if at all.

Photographs of the system are shown in Figure A- 1 to Figure A- 5.

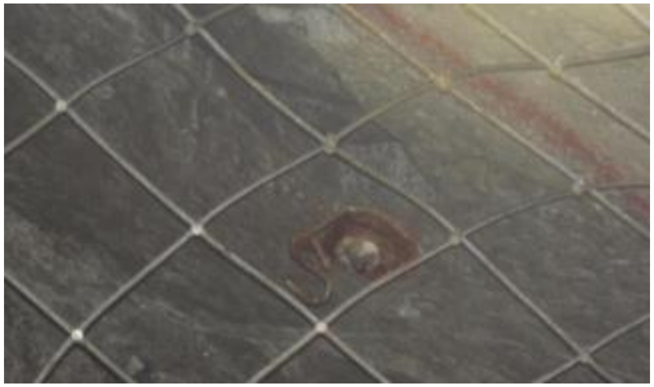

Figure A- 1: Net attachment to hanging-wall ("S")

Figure A- 2: Net attachment to hanging-wall (RYHP)

Figure A- 3: In-stope configuration (face area)

Figure A- 4: Drop test facility

Figure A- 5: Net performance by drop testing

Appendix A-2. Steel rope netting without tendons

Deep level mining in high stress, seismically susceptible, tabular hard rock environment using conventional (manual) stoping methods in ≤ 1.5 m stoping height

Overview

This support system is essentially a replica of the first system (Appendix A1) with the specific difference of being installed where tendon support is not incorporated into the system. The mesh is attached to the rock by way of timber packs only.

The yield and load bearing capacities of the system with tendons is similar to the one without tendons, as a result ratings of 8 and 9 were used for yield and load capacities respectively using the same reasoning as in Appendix A-1. Since this system does not incorporate tendons the initial stiffness is very low. The initial stiffness in this system has been rated as 1, which is lower than the system with tendons.

Tendons were not used and the tensioning of the steel rope net was done by hand resulting in a sagging installation. This generally meant that the quality of installation was less effective when compared to the system using tendons (Appendix A1) and a rating of 6 was given. During observations, there were no indications of blast and equipment damage on the netting; as a result both equipment and blast damage performance factors were rated as 9. The steel rope net had superficial rusting and this resulted in a corrosion rating of 8. The overall performance rating was determined as equal to the lowest performance factor and was rated as an 8.

The installation of steel rope netting is done by hand and no equipment is transported into the stopes, therefore a rating of 10 was given for equipment handling. The steel rope netting is flexible and can be easily rolled and transported on the scraper rope, resulting in easy material handling, hence a rating of 8. The installation of the steel rope nets is highly labour intensive and does not involve any mechanisation; therefore labour requirements got a rating of 2.5. The steel rope net without tendons was fairly easy to install, this meant an installation rating of 8.

Design methodology

The design methodology echoes that for system A1. The decision to install steel rope netting is based on similar criteria with the difference that stoping is carried out in a reduced height ($\leq$ 1.5 m) stope due to an alternative extraction cut for the orebody. In this case, a protective quartzite beam is retained in the hanging-wall to prevent a shale layer from being exposed. The protective beam is approximately 0.6 m thick and similarly fractured as in the case where the full cut includes removal of the shale layer. In this case, the installation of tendons is precluded due to the cut leading to excessive fracturing of the hanging-wall quartzite layer which, together with the shale layer that is retained above the quartzite beam, renders tendon installation ineffectual.

Photographs of the system are shown in Figure A- 6 to Figure A- 8.

Figure A- 6: Net retaining broken hanging without rock bolts

Figure A- 7: Perspective view of the net without bolts

Figure A- 8: Failed and rusted net in back areas

Appendix A-3. Stope shotcrete UG 2 stoping (low stope width < 2.2 m)

Shallow depth, hard rock mechanised room – and – pillar operation, where shotcrete is applied using hand-held pneumatic equipment

Ryder & Jager, (2002) describe unreinforced shotcrete as having good initial stiffness, poor yield capacity and medium load bearing capacity. The description was reflected in the ranking by assigning ratings of 9, 1 and 5 for initial stiffness, yield capacity and load bearing capacity respectively.

The shotcrete thickness was checked concurrently with application through probe holes and there is documentation of quality control processes (i.e. cube tests). This ensures good quality installation and as a result a rating of 9 was used. Blast performance was not observed on the mining face, however based on experience, shotcrete has been able to resist damage. There were no signs of shotcrete damage due to equipment. Due to the nature and composition of shotcrete it is not subjected to corrosion. In light of the above, blast, equipment and corrosion performance were rated as 9,9 and 10 respectively.

Shotcrete machine and raw materials (cement and sand) are transported fairly easily in an utility vehicle or LHD. However large amounts of cement and sand need to be transported to the working faces. Ratings of 8 and 6 were consequently assigned to equipment and materials handling. The application of shotcrete is semi-mechanised and requires very few people, resulting in a labour requirement rating of 8. The installation of shotcrete is fairly easy and was rated as 8.

Design methodology

- Shotcrete is applied as permanent areal support in areas where the ground has deteriorated. Ground conditions are assessed visually and the RMR system is used to determine the Trigger Action Response Plan (TARP) level. If the ground conditions (risk) is triggered then a remedial process is escalated to the level of responsibility required to deal with the risk
- In this particular case, the Rock Engineer assesses the risk and recommends shotcrete application as well as its thickness. No quantifiable systematic support design verification using design methods such as the Q-system was observed to be used.
- It was observed that shotcrete was applied consistently for special ground conditions such as potholes, prominent low-lying joints and dykes.
- Quality assurance monitored the strength of the shotcrete against curing time using the cube tests. The thickness of the shotcrete is checked via probe holes that are on 1 m × 1 m pattern.

Practical description of the system

In ground conditions which show visible deterioration, shotcrete is applied as permanent secondary support. The blast performance of the shotcrete could not be evaluated given its position behind the support. Resin grouted rebars are used as the primary support, and where borehole camera surveys identify gravitational block failure hazards, cable anchors are also used as additional support.

The dry ingredients (cement and sand) are placed into a hopper and then conveyed pneumatically through a hose to the nozzle. The nozzleman (i.e. the person who applies the shotcrete) controls the addition of water and air at the nozzle. The water and the dry mixture are not completely mixed - the mixing process is completed on impact on the rock surface.

Benefits of using shotcrete with tendon support include:

- There is an increase in fracturing in the vicinity of potholes and dykes, accompanied by sympathetic jointing, thereby increasing the chances of small rockfalls which cannot be controlled by tendons. Therefore, shotcrete is used to provide the areal support between the tendons.
- Shotcrete provides permanent areal support for workers, which is critical for bord and pillar operations, since the bords are continuously used as access, for the duration of a mining operation.
- Shotcrete adheres to the rock surface, thereby making it an ideal form of areal support in mechanised environments because mobile machinery cannot pull and rip it apart like mesh.

Limitations of shotcrete support:

- Since it was dry mix applied, it creates a lot of dust thereby increasing operation hazards.
- Large bulks of materials i.e. cement and sand need to be transported to locations where it will be applied, thereby presenting logistic issues.
- It obscures the visibility of discontinuities and other geological features as well as rock bolts, making it impossible to see deterioration over time.

Photographs of the system are shown in Figure A- 9 to Figure A- 12.

Figure A- 9: Dyke conditions to be shotcreted

Figure A- 10: Bulk shotcrete material

Figure A- 11: Shotcrete application procedure

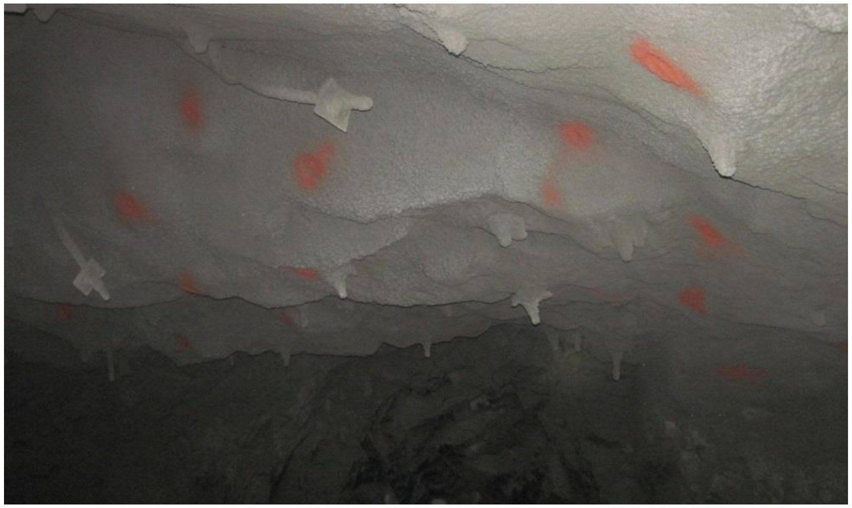

Figure A- 12: Dyke condition after shotcreting

Appendix A-4. Shotcrete: Merensky decline (high stope width > 3 m)

Shallow depth, hard rock mechanised access development decline tunnel, where shotcrete is applied using hand-held pneumatic equipment

Overview

The application of shotcrete in the Merensky decline is similar to the one in the UG 2 mining environment; except that this was in a development decline tunnel so the mining height was significantly different. Shotcrete application is undertaken due to the long-term nature of the excavation as well as expected weathering due to the shallow depth of the decline.

Shotcrete had been applied from the portal entrance up to the decline face. The design methodology was not specifically different from the UG 2, and the same shotcrete thickness of 50 mm was applied.

The benefits and limitations observed in the UG 2 are similarly applicable in the decline, with the added height limitation. This resulted in more material rebound and difficulty in determining that the correct shotcrete thickness had been applied.

Ryder & Jager, (2002) describe unreinforced shotcrete as having good initial stiffness, poor yield capacity and medium load bearing capacity. The description was reflected in the ranking by assigning ratings of 9, 1 and 5 for initial stiffness, yield capacity and load bearing capacity respectively.

The shotcrete thickness was checked concurrently with application through probe holes and there is documentation of quality control processes (i.e. cube tests). This ensures good quality installation and as a result a rating of 9 was used. Blast performance was not observed on the mining face, however based on experience, shotcrete has been able to resist damage. There were no signs of shotcrete damage due to equipment. Due to the nature and composition of shotcrete it is not subjected to corrosion. In light of the above, blast, equipment and corrosion performance were rated as 9, 9 and 10 respectively.

The shotcrete machine and raw materials (cement and sand) are transported fairly easily in an utility vehicle or LHD. However large amounts of cement and sand need to be transported to the working faces. Ratings of 8 and 5 were consequently assigned to equipment and materials handling. It should be noted that in the decline larger amounts of raw materials are handled compared to the stoping environment, therefore a lower rating for the materials handling. The application of shotcrete is semi-mechanised and requires very few people. However, due to the increased quantities that need to be applied and carried, there is a larger labour complement in the decline compared to the stopes and therefore labour requirements are rated 8. The installation of shotcrete is fairly easy and was rated as 8.

Appendix A-5. Steel welded mesh Ikamva (Kloof 4shaft)

Deep level mining in high stress, seismically active, tabular hard rock environment using conventional (manual) stoping methods

Overview

Ikamva (Kloof 4shaft) shaft is a conventional ultra-deep gold mine which has a highly fractured hanging wall. The support system is installed in gullies and raise lines and it comprises friction based tendons (Hydrabolts) and steel welded mesh.

The weld mesh is made from steel that has steel strength of between 400 – 600 MPa with strand diameters of 4 mm. This gave strand tensile strengths of 5.0 – 7.5 kN. These strand strengths are lower when compared to other meshes (i.e. thicker diameter welded mesh, ultra – strength chain - link mesh and steel rope netting).

Ryder & Jager, (2002) describe wire mesh and lace as having poor initial stiffness, good yield capacity and low load bearing capacity. The descriptions reflect the characteristics of the weaker chain - link mesh. In contrast, the weld mesh used at Ikamva is on the other end of the spectrum as it much stronger than the chain – link mesh. Laboratory test results from literature have shown that weld mesh has higher load bearing capacities, greater stiffness and fairly good yield (deformation) capacity although it is lower than chain – link mesh. Considering the above, ratings of 6, 4 and 5 were assigned for initial stiffness, yield capacity and load bearing capacity respectively.

The observed mesh overlaps between adjacent mesh panels, as well as the orientation of the mesh strands, suggested that the installation procedures and standards were adhered to. This ensures good quality installation and as a result a rating of 8 was assigned for quality of installation.

Blast – on performance and equipment damage performance of the mesh was not observed directly. However, the predicted blast - on and equipment resistance performance of mesh was based on the performance of a larger strand diameter weld mesh installed in almost similar conditions. The mesh is expected to have moderate damage due to both blast and equipment action, which can be rectified by rehabilitation of the mesh. Blast and equipment performance ratings of 6 and 5 were assigned respectively. The mesh is zinc coated and is expected to have superficial rusting at worst. A corrosion performance rating of 9 was therefore assigned.

The mesh is largely installed by hand. Airlegs for drilling of Hydrabolt holes, and small water pumps, are the only equipment items that are transported into the stopes for mesh installation. This equipment is easy to transport into the working face, therefore an equipment handling rating of 9 was assigned.

The mesh and Hydrabolts are transported from surface in rail bound material cars up to the stope entrance. They are then transported on scraper ropes from the stope entrance into the working area, the transportation is much more difficult and has been rated 5.

The mesh installation is labour intensive, fairly easy, conventional and does not require specialized skill, consequentially labour requirements and installation are rated 2.5 and 5 respectively.

Design consideration

- Support performance: mesh tests are not done on-mine, however the supplier carries out tests and subsequently provides the QA and QC documentation to the mine.
- Friction tendons (hydrabolts) are used in conjunction with steel welded mesh in the gullies and / or raises. Steel welded mesh is held in place by tendon bearing plates.
- The tendon support performance and specifications are guided by the empirical evaluation of potential rockfalls, based on available fall of ground (FoG) records.
- The hanging wall of the gully is characterised by brittle stress fractured rock mass. The fractured rock mass increases the risk of injury and FoGs, therefore using mesh to mitigate the risk is a crucial consideration.

Practical description of the system

Permanent steel welded mesh is installed on the reef development horizon (i.e. gullies and / or raises) and with friction tendons (hydrabolts) to provide areal coverage up to the face. Once barring down has been completed in the gully and the work place made safe, the mesh is fastened against the hanging wall by way of camlock props. Tendons are then installed and subsequently camlock props removed, thereby comprising permanent areal support of steel welded mesh and friction tendons.

Benefits of using welded mesh with tendons include:

- Excellent retention of rockfalls, both gravitational and seismic shakedown.
- Addresses limitations of tendon support by providing areal coverage for a brittle, severely stress-fractured hanging-wall.
- Removal of areal support prior to each blast is not required, which saves time and reduces worker exposure.
- Permanent protection for workers and limitation of falls of ground (logistics and production restrictions) in the back area and face area is provided resulting in improved safety performance for the mine workers and limited interruptions therefore improvement to productivity.

Limitations of the steel welded mesh include:

- Labour intensive.
- Strand thickness (gauge), aperture and net span (area) are directly related to the weight of the material and how unwieldy the mesh is to manoeuvre.
- Due to the varying profile of the rock mass as well as stiffness of the mesh, it is considered an arduous task to achieve total confinement of the rock mass using welded mesh - often results in a sagging mesh installation.
- Interference with equipment (damage).
- Low level design considerations. Selection of the steel welded mesh is largely driven by empirical (experiential) input, professional opinion and very little laboratory type drop testing is reported.

Photographs of the system are shown in Figure A- 13 to Figure A- 15.

Figure A- 13: Good weld mesh condition in a gully and hydrabolts

Figure A- 14: Weld mesh retaining broken rock

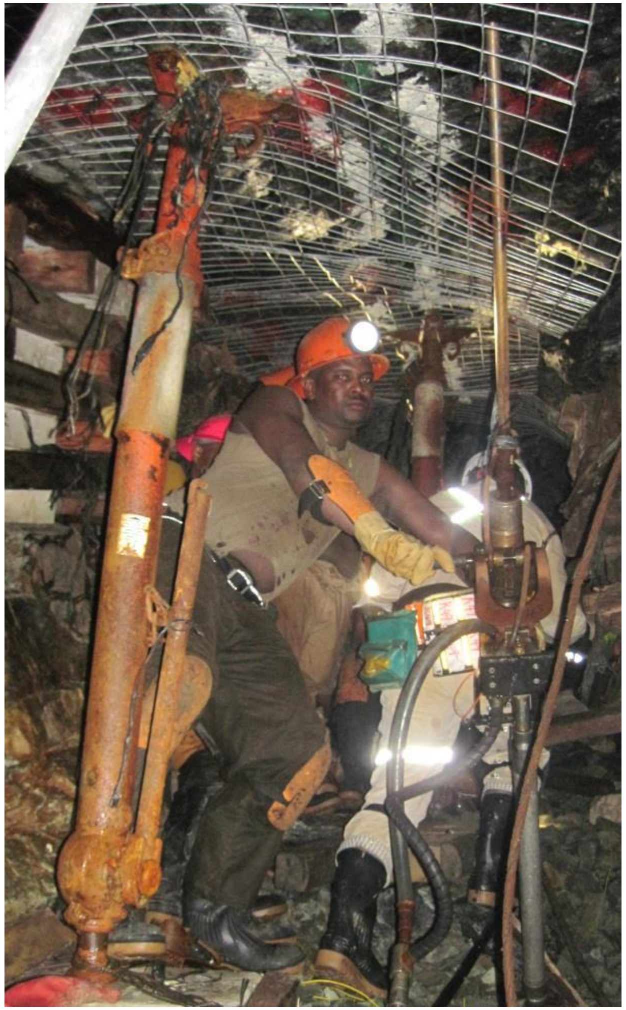

Figure A- 15: Weld mesh installation

Appendix A-6. Steel welded mesh Tau Tona

Deep level mining in high stress, seismically susceptible, tabular hard rock environment using conventional (manual) stoping methods

Overview

Tau Tona is a conventional ultra-deep gold mine which has a highly fractured hanging wall. The permanent support system is installed in gullies and raise lines and it comprises friction based tendons (split sets) and steel welded mesh.

The weld mesh is made from steel that has steel strength of between 400 – 600 MPa with strand diameters of 5.6 mm. This gave strand tensile strengths of 9.9 – 14.8 kN. These strand strengths are lower when compared to steel rope netting, and compare well with the observed ultra – strength chain link mesh.

Ryder & Jager, (2002) describe wire mesh and lacing as having poor initial stiffness, good yield capacity and low load bearing capacity. The descriptions reflect the characteristics of the weaker chain - link mesh. In contrast, the weld mesh used at Tau Tona is similar to that used at Ikamva shaft and is significantly stiffer than the chain – link mesh. Laboratory test results from literature have shown that when compared to chain link mesh, steel weld mesh has higher load bearing capacities, greater stiffness and fairly good yield (deformation) capacity although it is lower than chain – link mesh.

The mesh used at Tau Tona has larger strand diameters than that used at Ikamva and is expected to be a lot more robust i.e. greater load bearing capacities, higher stiffness and better yield capabilities. In light of the discussion, ratings of 7, 6 and 7 were assigned for initial stiffness, yield capacity and load bearing capacity respectively.

The observed mesh overlaps between adjacent mesh panels as well as the orientation of the mesh strands suggested that the installation procedures and standards were adhered to. This ensured good quality installation and as a result a rating of 9 was assigned for quality of installation.

The mesh had moderate damage (which can be rectified by rehabilitation) due to both blast and equipment action; however, equipment damage to the mesh is greater. Blast and equipment performance ratings of 7 and 5 were assigned respectively. The mesh is zinc coated and it showed superficial rust. A corrosion performance rating of 9 was therefore assigned.

The mesh is largely installed by hand. Airlegs for drilling of split-set holes is the only equipment that is transported into the stopes for mesh installation. This equipment is easy to transport into the working face, therefore an equipment handling rating of 9 was assigned.

The mesh and split-sets are transported from surface in rail bound material cars up to the stope entrance. They are then transported on scraper ropes from the stope entrance into the working area. This transportation of which is much more difficult than the mesh at Ikamva, due to the increased weight per unit area, and the mesh is stiffer (more difficult to install) thus it was rated 4.

The mesh installation is labour intensive, fairly easy, conventional, and does not require specialized skill and consequentially labour requirements and installation are rated 2.5 and 5 respectively.

Design consideration

- Support performance: mesh tests are not done on-mine, however the supplier carries out tests and subsequently provides the QA and QC documentation to the mine.
- Friction tendons (Split - sets) are used in conjunction with steel welded mesh in the gullies and / or raises. Steel welded mesh is held in place by tendon bearing plates.
- The tendon support performance and specifications are guided by the empirical evaluation of potential rockfalls, based on available fall of ground (FoG) records.

- The hanging wall of the gully is characterised by brittle stress fractured rock mass. The fractured rock mass increases the risk of injury and FoGs, therefore using mesh to mitigate the risk is a crucial consideration.

Practical description of the system

Permanent steel welded mesh is installed on the reef development horizon (i.e. gullies and / or raises) and with friction tendons (split sets) to provide areal coverage up to the face. Once barring down has been completed in the gully and the work place made safe, the mesh is fastened against the hanging wall by way of camlock props. Tendons are then installed and subsequently camlock props removed, thereby comprising permanent areal support of steel welded mesh and friction tendons.

Benefits of using welded mesh with tendons include:

The benefits offered by the welded steel mesh at Tau Tona are similar to those at Ikamva:

- Excellent retention of rockfalls, both gravitational and seismic shakedown.
- Addresses limitations of tendon support to retain a brittle, severely stress-fractured hanging-wall.
- Removal of areal support prior to each blast is not required, which saves time and reduces worker exposure.
- Permanent protection for workers and limitation of falls of ground (logistics and production restrictions) in the back area and face area is provided resulting in improved safety performance for the mine workers and limited interruptions (therefore improvement) to productivity.

Limitations of the steel welded mesh include:

- Labour intensive.
- Strand thickness (gauge), aperture and net span (area) are directly related to the weight of the material and how unwieldy the mesh is to manoeuvre.
- Due to the varying profile of the rock mass as well as stiffness of the mesh, it is considered an arduous task to achieve total confinement of the rock mass using welded mesh. This often results in a sagging mesh installation.

- Interference with equipment (damage).
- Low level design considerations. Selection of the steel welded mesh is largely driven by empirical (experiential) input, professional opinion and very little laboratory type drop testing is reported.

Photographs of the system are shown in Figure A- 16 to Figure A- 18.

Figure A- 16: Good weld mesh condition in a gully

Figure A- 17: Weld mesh damage caused by scrapers

Figure A- 18: Weld mesh damage caused by blasting

Appendix A-7. Steel welded mesh Dishaba shaft

Shallow to intermediate level mining in jointed and static, tabular hard rock environment using conventional (manual) stoping methods

Overview

Dishaba shaft is a conventional shallow to intermediate depth mine which has a jointed hanging wall. The areal support system is installed in gullies on an ad hoc basis and it comprises cable anchors and steel welded mesh.

The weld mesh is made from steel that has steel strengths of between 400 – 600 MPa with strand diameters of 4 mm. This gave strand tensile strengths of 5.0 – 7.5 kN. These strand strengths are lower when compared to other meshes (i.e. thicker diameter welded mesh, ultra – strength chain - link mesh and steel rope netting).

Ryder & Jager, (2002) describe wire mesh and lace as having poor initial stiffness, good yield capacity and low load bearing capacity. The descriptions reflect the characteristics of the weaker chain - link mesh. In contrast, the weld mesh used at Dishaba is much stiffer than the chain – link mesh. Laboratory test results from literature have shown that, when compared to chain link mesh, weld mesh has higher load bearing capacities, greater stiffness, fairly good yield (deformation) albeit being lower than chain – link mesh. In light of the above, ratings of 6, 4 and 5 were assigned for initial stiffness, yield capacity and load bearing capacity respectively.

The observed mesh overlaps between adjacent mesh panels in places were not in line with the installation procedures and standards. This resulted in “not so good” quality installation and as a result a rating of 5 was assigned for quality of installation.

The mesh had moderate damage (which can be rectified by rehabilitation) due to both blast and equipment action; however the equipment damaged the mesh more. Blast and equipment performance ratings of 6 and 3 were assigned for blast and equipment damage respectively. The mesh was installed in a corrosive environment and the mesh showed severe rusting, and corrosion performance was rated 5.

The mesh is largely installed by hand. Airlegs for drilling of cable anchor holes and small grout pumps, are the only equipment that is transported into the stopes for mesh installation. This equipment is easy to transport into the working face, therefore an equipment handling rating of 9 was assigned.

The mesh and cable anchors are transported from surface in rail bound material cars up to the stope entrance. They are then transported on scraper ropes from the stope entrance into the working area, the transportation of which is difficult and has been rated 5.

The mesh installation is labour intensive, fairly easy, conventional, and does not require specialized skill. Consequentially, labour requirements and installation are rated 2.5 and 5 respectively.

Design consideration

- Support performance: mesh tests are not done on-mine. However, the supplier carries out tests and subsequently provides the QA and QC documentation to the mine.
- Cable anchors are used in conjunction with steel welded mesh in the gullies and / or raises. Steel welded mesh is held in place by tendon bearing plates.
- The tendon support performance and specifications are guided thorough the empirical evaluation of potential rockfalls, based on available fall of ground (FoG) records.
- The hanging wall of the gully is characterised by jointed rock mass. The jointed rock mass increases the risk of injury and FoGs, therefore using mesh to mitigate the risk is a crucial consideration.

Practical description of the system

Permanent areal support is installed on the reef horizon and in gullies. Cable anchors are installed as the primary support units. The steel welded mesh is installed as secondary support. Mesh is connected to cable anchor bearing plates using S-hooks in places or a secondary bearing plate configuration.

Benefits of using welded mesh with tendons include:

The benefits offered by the welded steel mesh at Dishaba are similar to those at Ikamva and Tau Tona:

- Excellent retention of rockfalls, both gravitational and seismic shakedown.
- Addresses limitations of tendon support to retain a brittle, severely stress-fractured hanging-wall.
- Removal of areal support prior to each blast is not required, which saves time and reduces worker exposure.
- Permanent protection for workers and limitation of falls of ground (logistics and production restrictions) in the back area and face area is provided, resulting in improved safety performance for the mine workers and limited interruptions (therefore improvement) to productivity.

Limitations of the steel welded mesh include:

- Labour intensive.
- Strand thickness (gauge), aperture and net span (area) are directly related to the weight of the material and how unwieldy the mesh is to manoeuvre.
- Due to the varying profile of the rock mass as well as stiffness of the mesh, it is considered an arduous task to achieve total confinement of the rock mass using welded mesh. This often results in a sagging mesh installation.
- Interference with equipment (damage).
- Low level design considerations. Selection of the steel welded mesh is largely driven by empirical (experiential) input, professional opinion and very little laboratory type drop testing is reported.

Photographs of the system are shown in Figure A- 19 to Figure A- 21.

Figure A- 19: Blast damaged mesh still retaining rock

Figure A- 20: Scraper damaged mesh

Figure A- 21: Heavily rusted mesh connected to anchors by s - hooks

Appendix A-8. Chain link (diamond) meshing

Shallow depth, hard rock environment using conventional (manual) stoping methods

Overview

Areal support installation was observed in the only trial panel on the mine and this exploited the UG 2 reef. The hangingwall of the UG 2 is a 40 cm thick beam that resulted from the presence of chromitite stringers in the overlying pyroxenite. The mining standard is to mine out the hanging wall beam since it poses a risk of FoG if it is undercut. However, extracting the beam and the ore results in dilution and reduced ore grades. In an attempt to minimise ore dilution the beam was undercut and the risk of FoG was managed by the installation of ultra-strength chain link mesh with tendons.

The observed mesh is made of ultra-strength steel mesh and had a strand tensile strength of 12.5 kN which is exceptionally high for 3 mm diameter wire. Based on the relatively high deformation capabilities of the mesh, it is expected to have high yielding capabilities. The design of the mesh is such that it can only be rolled in one direction and is relatively stiff in the other direction; this together with the rockbolt systems ensures good initial stiffness. Based on the above, ratings of 7, 8 and 8 were assigned for initial stiffness, yield capacity and load capacity respectively.

The mesh is installed relatively taut against the hangingwall, albeit tensioned manually. The mesh was installed in a trial site, and had been installed for more than a year, when observations were carried out. It was supporting rock that had unravelled from the hanging, indicating good quality installation, and as a result, installation quality was rated 9.

The mesh was installed right up to the mining face and was blasted on during observations. No indications of blast damage were found. However, the mesh was moderately damaged by equipment and this damage could be corrected by rehabilitation. The stope in which the mesh had been installed had dripping water but the mesh showed no rusting. Consequently, blast performance, equipment damage and corrosion performance were rated 9, 6 and 10 respectively.

The chain-link mesh was used in conjunction with mechanical end anchors. These are installed by making use of airlegs. The transportation of airlegs in the stopes is very easy and as such, equipment handling was rated 9. The mesh can be easily rolled in one direction and is fairly light to carry, therefore material handling is rated 8. The installation of the mesh does not require a large labour complement but it is not mechanised. As a result, a labour requirement rating of 5 was assigned. The mesh is fairly easy to install due to its ability to be rolled in one direction, thus ensuring it is installed correctly. Adjacent mesh panels are connected by use of clips, which is relatively easy to install, and this led to an installation rating of 7.

Design consideration

- Support performance: the performance of the ultra-strength mesh is determined through laboratory tests. Testing was carried out to simulate rock burst scenarios (through drop testing methods) and quasi-static conditions by loading the mesh by way of a hydraulic ram. Load deformation curves are then plotted from which performance parameters can be derived
- Support specifications (resistance): The support specification is determined from the demand applied on the support system by the rock mass. The demand is determined through the empirical evaluation of potential rockfalls, based on likely FoG incidents. The support resistance is then compared to the support performance.

Safety: additional support design was necessary to mitigate the risk, associated with leaving the beam in the stope hangingwall.

Benefits of ultra-strength chain link mesh include:

- Excellent retention of rockfalls both gravitational (observed on-mine) and seismic determined experimentally.
- The mesh was installed up to the face and was subjected to blast loading and the mesh performed exceedingly well.
- The mesh is relatively light and can be folded, thus making it easier to handle compared to welded mesh.
- The mesh is flexible in one direction and is stiff in the other direction. Consequently, it has the benefit offered by traditional chain link mesh and relatively greater stiffness offered by welded mesh. Displacement of the mesh is therefore minimised when loaded.

- The installation of mesh eliminates the need for safety net installation, and this improves the overall support installation cycle time.

Limitations of the ultra-strength chain link mesh include:

- Sagging may occur given that the mesh is tensioned manually. Sagging may subsequently allow rock to deflect the mesh when rockfalls occur. These incidents may result in injuries to personnel or may result in disruptions to the serviceability of excavations.
- The cost of the mesh is considered to be moderate – which may deter use on a routine basis.
- Ultra-strength mesh remains susceptible to damage from machinery in the stope.

Photographs of the system are shown in Figure A- 22 to Figure A- 25.

Figure A- 22: Ultra-strength chain link mesh installation

Figure A- 24: Chain link mesh retaining broken hanging.

Figure A- 23: Scraper damage on chain link

Figure A- 25: Blasted-on mesh with steel rope to absorb blast impact

Appendix A-9. TSL in UG 2 stoping environment

Shallow depth, hard rock mechanised room – and – pillar operation, where TSL is applied using hand-held pneumatic equipment

Overview

Observations were carried out at a mine on the southern part of the eastern limb of the Bushveld complex. The geological sequence present comprises the Upper Critical Zone and the lower part of the Main Zone of the Bushveld Complex; as a result two economically viable reefs are mined for PGEs and these are the Merensky and UG 2 reefs. At the time of the observations, the mine was only exploiting the UG 2 reef.

There are several geological and geotechnical domains that can be encountered underground. A portion of a mine where similar geological conditions exist. This gives rise to a unique set of identifiable rock related hazards for which a common set of strategies can be employed, to minimise the risk resulting from mining. This portion of the mine is referred to as a Ground Control District (GCD).

From the exposures and experience gained at the mine, five elementary GCDs have been identified on the UG 2 and Merensky (although not being mined) reef horizons. One of the GCDs identified has been called the Pothole GCD. In the vicinity of potholes observations suggest that the dip of the reef becomes somewhat steeper as it is dragged down by the pothole. The chromitite stringer, which occurs above the reef, has also been observed to cut down towards the reef, resulting in a thinner hanging wall beam closer to the pothole. The probability and frequency of low angled features are higher in pothole areas as well. Furthermore, the contact between the hanging wall pyroxenite (HW1) and anorthosite (HW2) appears to be dragged down towards the reef, and anorthosite is known to be more brittle than pyroxenite.

These characteristics of a pothole may result in difficulties with managing the ground, and the primary support is not sufficient to control the ground. As a result, secondary areal coverage in the form of TSL is installed.

Ryder & Jager, (2002) did not specifically give descriptions of TSLs when they were describing support systems characteristics. However, due to the relatively similar behaviour between TSLs and shotcrete, TSL support characteristic descriptions and ratings were extrapolated from those of shotcrete. TSLs are expected to have good initial stiffness although relatively less than shotcrete, poor yield capacity, but slightly higher than shotcrete due to their flexibility, and medium load bearing capacity is similar to that of shotcrete. These descriptions are reflected in the ranking tool by assigning ratings of 8, 2 and 5 for initial stiffness, yield capacity and load bearing capacity respectively.

The applied thickness of TSLs was estimated visually during application based on experience. No rock exposures were observed in areas that had been sprayed, and the rock mass looked generally better after application. This ensured a fairly good quality installation and as a result a rating of 8 was assigned. Blast performance was not observed in the particular mining face. However, based on observations elsewhere, TSLs resisted blast impact very well. There were no signs of TSL damage due to equipment. Due to the nature and composition of TSL it cannot be subjected to corrosion. In light of the above, blast, equipment and corrosion performance were rated as 9, 8 and 10 respectively.

TSL machine and raw materials (cement and sand) are transported fairly easily in a utility vehicle or LHDs. However large amounts of raw materials need to be transported to the working faces, although relatively less than those of shotcrete. Rating of 8 and 6 were consequently assigned to equipment and materials handling respectively. The application of TSL is semi-mechanised and requires very few people, therefore labour requirements are rated 8. The installation of TSL is fairly easy and was rated as 8.

Design considerations

- Support specifications and performance: these are determined in the laboratory and presented as strength characteristics after 7 days of curing. There are not set out standards (guidelines) for the testing of TSLs.
- Support resistance and demand: there appears to be no set-out methodology to calculate the demand applied on the support. The TSL application has been done on a trial-and-error basis (based on personal experience of the respective mining personnel) and no rigorous scientific

approach is available to provide information regarding how the thickness of the TSL was determined.

Benefits of using TSL include:

- It is a wet mix, therefore the amount of dust produced is minimal when compared to other dry mix cementitious based support.
- The thickness of the TSL required to support a particular block size is relatively less than that required for shotcrete. This means that less material needs to be transported to the site.

Limitations of TSL include:

- There are no standard procedures to determine whether sufficient amounts of TSL have been applied, due to the fast setting nature of the TSL. Hence, the problem of under or over supporting (too thin or too thickly applied) arises.
- Visual estimation of TSL thickness is carried out in relation to the rock-bolt washer plates, i.e. if the TSL covers the washer plates the thickness is considered to be to the minimum requirement (8 mm).
- Like most other membrane systems, TSLs conceal geological structures and discontinuities, making it difficult for hazard / risk identification and implementing a response plan.

Photographs of the system are shown in Figure A- 26 to Figure A- 28.

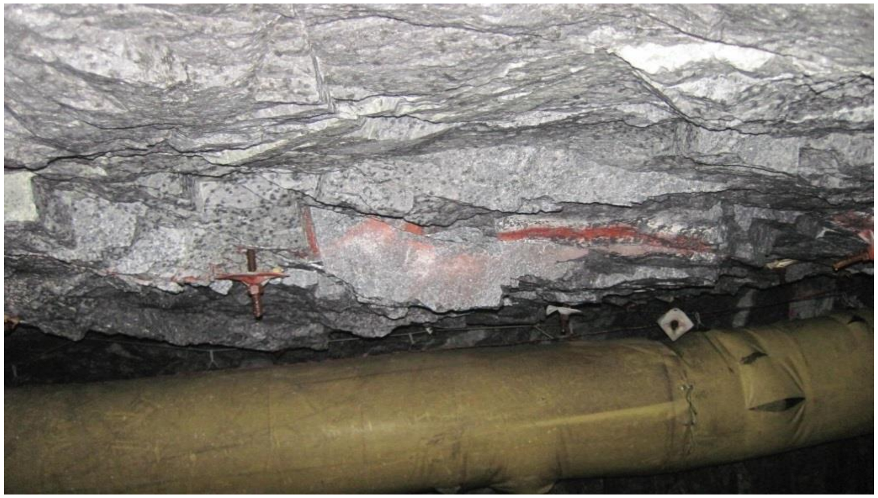

Figure A- 26: Rock mass condition at pothole edge.

Figure A- 27: Rock mass condition after TSL application.

Figure A- 28: TSL application.

Appendix A-10. TSL in UG 2 stoping environment

Shallow to medium depth, hard rock scattered conventional breast mining operation, where TSL is applied using hand-held pneumatic equipment

Overview

The mining operation is located on the north-western sector of the Bushveld Complex. The two reef bodies mined are the Merensky and the UG 2 Reefs. Observations were carried out in a UG 2 working area. The immediate hanging wall of the UG 2 is an altered olivine-rich poikilitic pyroxenite (harzburgite) layer which occurs at a maximum depth of 70 cm in the hanging wall. The altered nature of the pyroxenite results in time-dependant scaling of the hanging wall and the separation of 2 cm - 15 cm thick sheets of pyroxenite. These layers are relatively cohesionless and warrant beam building through the use of mechanical end anchors.

The UG 2 reef package often drops below its normal plane and this is referred to as 'reef slumping'. The hanging wall beam is then disrupted by the mining not being able to closely follow the reef top contact, resulting in higher risks of exposing the harzburgite and experiencing FOGs. The site where observations were carried out was a slumped UG 2 reef horizon centre raise development access. OSRO-straps are typically used as areal coverage in the centre raise hanging-walls on the operation; however, the straps are susceptible to machinery damage during cleaning as well as blast damage during ledging.

The design considerations follow those described in A7. The design approach was experiential and there was very little evidence of scientific and engineering design process inputs. A standard 8 mm thick liner was sprayed that visually appeared to perform well against rockfalls. Further evidence of the adequacy of the performance of the TSL was noted by the good blast-on performance of the TSL. The TSL was used in conjunction with mechanical end anchors as the primary support units.

Ryder & Jager, (2002) did not specifically give descriptions of TSLs when they were describing support system characteristics. However, due to the relatively similar behaviour of TSLs and shotcrete, TSL support characteristic descriptions and ratings were extrapolated from those of shotcrete. TSL is expected to have good initial stiffness although relatively less than shotcrete, poor yield capacity, but slightly higher than shotcrete, due to their flexibility, and medium load bearing capacity in the same range as shotcrete. These descriptions are reflected in the ranking tool by assigning ratings of 8, 2 and 5 for initial stiffness, yield capacity and load bearing capacity respectively.

The applied thickness of TSLs was estimated visually during application, based on experience. No rock exposures were observed in areas that had been sprayed, and were the thickness was less than the required thickness, it was marked off for respraying. Generally the rock mass looked better after application, and interactions with the workers confirmed this sentiment. The workers stated that they felt safer working under the TSL sprayed roof. This ensured a fairly good quality installation and as a result a rating of 8 was assigned. TSL blast on performance was observed in this particular mining face, and it resisted the blast impact very well. There were no signs of TSL damage due to equipment. Due to the nature and composition of TSL it cannot be subjected to corrosion. In light of the above, blast, equipment and corrosion performance were rated as 9, 8 and 10 respectively.

TSL machine, and raw materials (cement and sand), are transported with some difficulty in locos as well as by hand into the stopes. This together with the large amounts of raw materials that need to be transported to the working faces, resulted in rating of 5 and 4 assigned to equipment and materials handling respectively. The application of shotcrete is semi-mechanised and requires very few people, therefore labour requirements are rated 8. The installation of TSL is fairly easy and was rated as 8.

Benefits of using TSL include:

- In addition to the benefits described above, the TSL was found to have very little blast damage.
- The TSL was subjected to mechanical action of machinery, but did not fail as expected for mesh.
- The TSL machine was relatively small and could be easily transported by two persons, making the handling and logistics easy.

Limitations of TSL include:

- The limitations of TSL described in A7 are also applicable in A8. However, site specific problems were identified. It was observed that small blocks between tendons are well supported by the TSL. However, following the initial ledging blast, exposure and destabilisation of large wedges occurred. The TSL is unable to support large wedges against failure. It is for this purpose that the tendon support is necessary. Barring down of relatively bigger loose rocks in the ledging panels extended into the raise line, and subsequently the barred rock and the bonded TSL are detached from the hangingwall, resulting in localised patches of no areal coverage.
- It was difficult to measure the thickness of the TSL whilst spraying, due to the fast setting nature of the TSL. As a consequence, some places were barely covered with the right amount of TSL.

Photographic illustrations of the system are presented in Figure A- 29 to Figure A- 32.

Figure A- 29: TSL condition in gully.

Figure A- 30: Scraper rope carved hanging wall.

Figure A- 31: TSL damage due to large block.

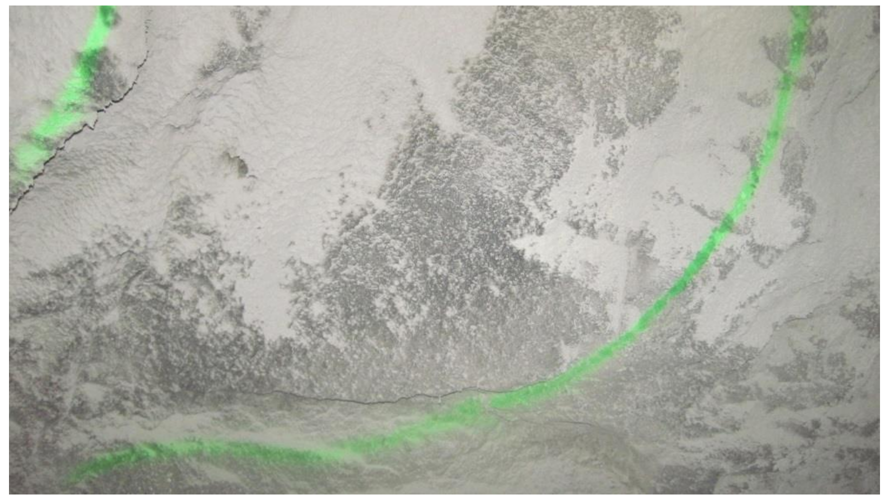

Figure A- 32: Insufficient TSL thickness application

Appendix B

Assumptions and corrections used alongside the ranking tool

Where testing data of support units in the ranking tool (observed underground) is absent, the most similar support units were used. However, corrections and/or assumptions have been applied to the test results to be applicable to the observed data. The assumptions and corrections done on the test results are described in Table B-1 and Table B-2. The corrected assumptions are presented in the ranking tool with blue shading.

Table B-1: Assumptions and corrections in the assignment of ranking values for support performance – for Ortlepp and Stacey test results

B-1. Potvin Corrections

- Kloof 4 shaft and Dishaba mesh were similar to the ones tested by Ortlepp and Stacey (1997) therefore the test results have been used without adjustments

- Booysendal applied 50 mm of unreinforced shotcrete which is similar to the shotcrete tested by Ortlepp and Stacey (1997) therefore the test results also used without adjustments

- Tau Tona (5.6 mm) mesh is similar to Kloof 4 shaft and Dishaba (4 mm) mesh with the only difference being the diameter of the mesh. The following calculations were done to estimate the deformation and energy capacity of the Tau Tona mesh

a) Peak load (strand strength) of Tau Tona mesh will increase by a factor of 1.96 based on the relationship:

Load = Steel strength × Strand area, with diameter being the only variable

Load increase factor = (5.6/4)^2 = 1.96

b) An estimate of the displacement increase factor due to change in diameter was based on WASM static test results. It was found that a decrease from 4 mm diameter to 3 mm resulted in approximately 0.74 deformation decrease. Therefore an increase from 4 mm to 5.6 mm will result in a displacement factor of 1.38

c) Energy is function of load (peak load) and displacement based on the following equation

Energy = Force × displacement

Energy increase factor = 1.38 × 1.96 = 2.7

- 80 mm by 3.0 mm ultra-strength mesh is used at Karee 4 Belt, therefore results of the 75 mm by 3.2 mm chain link mesh were used to estimate the performance of the mesh.

a) Peak load (strand strength) of Karee 4 Belt mesh will increase by a factor of 2.83 based on the relationship:

Load = Steel strength × Strand area, with diameter and steel strengths being the variables

Load increase factor = (1770/550) × (3/3.2)^2 = 2.83

b) The relationship between strength and deformation has not been established but is expected to be greater than the one determined from the tests, the deformation factor is k.

c) Energy = Force × displacement

Energy increase factor = 2.83 × k = 2.83 k
where k > 1

Table B-2: Assumptions and corrections in the assignment of ranking values for support performance - Canadian Handbook for Rockbursts (CHR)

B-2. Canadian Handbook for Rockbursts (CHR)

3.8 mm and 5.8 mm diameter mesh were tested by Kaiser et al (1996), it was assumed that 0.2 mm will not affect the results. Therefore 3.8 mm mesh results were used Kloof 4 shaft and Dishaba mesh (both which are 4.0 mm) whilst 5.8 mm results were used for Tau Tona mesh which has a 5.6 mm diameter. The results used for the Kloof, Dishaba and Tau Tona are as reported by Kaiser et al (1996) without adjustments.

3.0 mm ultra-strength mesh is used at Karee 4 Belt, therefore results of the 3.8 mm chain link mesh were used to estimate the performance of the mesh.

a) Peak load (strand strength) of Karee 4 Belt mesh will increase by a factor of 2.0 based on the relationship:
Load = Steel strength × Strand area, with diameter and steel strengths being the variables
Load increase factor = (1770/550) × (3/3.8)^2 = 2.0

b) The relationship between strength and deformation has not been established but is expected to be greater than the one determined from the tests, the deformation factor is k.

c) Energy = Force × displacement
Energy increase factor = 2.0 × k = 2.0 k
where k > 1

Table B-3: Assumptions and corrections in the assignment of ranking values for support performance (Western Australian School of Mines (WASM) results)

B-3. Western Australian School of Mines (WASM) results

- Tau Tona (5.6 mm) mesh is similar to 5.6 mm weld mesh that was tested at the WASM test facility, therefore the test results have been used without adjustments

- Kloof 4 shaft and Dishaba (4 mm) mesh is similar to Tau Tona (5.6 mm) mesh with the only difference being the diameter of the mesh. The following calculations were done to estimate the deformation and energy capacity of the Kloof 4 shaft and Dishaba mesh:

a) Peak load (strand strength) of the mesh will decrease by a factor of 0.51 based on the relationship:
  Load = Steel strength × Strand area, with diameter being the only variable
  Load increase factor = (4/5.6)^2 = 0.51

b) An estimate of the displacement increase factor due to change in diameter was based on WASM static test results. It was found that a decrease from 4 mm diameter to 3 mm resulted in approximately 0.74 deformation decrease. Therefore a decrease from 5.6 mm to 4 mm will result in a displacement factor of 0.72

c) Energy is function of load (peak load) and displacement based on the following equation
  Energy = Force × displacement
  Energy increase factor = 0.51 × 0.72 = 0.37
- 80 mm by 3.0 mm ultra-strength mesh is used at Karee 4 Belt, therefore results of the 75 mm by 3.2 mm chain link mesh were used to estimate the performance of the mesh.

a) Peak load (strand strength) of Karee 4 Belt mesh will increase by a factor of 2.83 based on the relationship:
  Load = Steel strength × Strand area, with diameter and steel strengths being the variables
  Load increase factor = (1770/550) × (3/3.2)^2 = 2.83

b) The relationship between strength and deformation has not been established but is expected to be greater than the one determined from the tests, the deformation factor is k.

c) Energy = Force × displacement
  Energy increase factor = 2.83 × k = 2.83 k
  where k > 1