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# ABBREVIATIONS, NOMENCLATURE and SI UNITS

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_c$</td>
<td>Adhesion capacity</td>
</tr>
<tr>
<td>CoP</td>
<td>Code of Practice</td>
</tr>
<tr>
<td>$D_{20}$</td>
<td>Displacement at 20 % of design load</td>
</tr>
<tr>
<td>dbs</td>
<td>depth below surface</td>
</tr>
<tr>
<td>$\delta_d$</td>
<td>Quasi-static displacement</td>
</tr>
<tr>
<td>$D_d$</td>
<td>Design load displacement</td>
</tr>
<tr>
<td>DMR</td>
<td>Department of Mineral Resources</td>
</tr>
<tr>
<td>$D_0$</td>
<td>Intersection of the line used to calculate the stiffness and the x-axis</td>
</tr>
<tr>
<td>FoS</td>
<td>Factor of Safety (Ratio of capacity vs demand)</td>
</tr>
<tr>
<td>FRS</td>
<td>Fibre-reinforced shotcrete</td>
</tr>
<tr>
<td>GSI</td>
<td>Geological Strength Index</td>
</tr>
<tr>
<td>$K_s$</td>
<td>Screen stiffness</td>
</tr>
<tr>
<td>kN</td>
<td>kiloNewton (SI unit)</td>
</tr>
<tr>
<td>$L_{20}$</td>
<td>20% of the design load</td>
</tr>
<tr>
<td>$L_d$</td>
<td>Design load</td>
</tr>
<tr>
<td>$m_d$</td>
<td>Moment (demand), kNm.m$^{-1}$</td>
</tr>
<tr>
<td>MHSC</td>
<td>Mine Health and Safety Council</td>
</tr>
<tr>
<td>$\sigma_a$</td>
<td>Adhesive strength (lab test results), MPa</td>
</tr>
<tr>
<td>$\sigma_p$</td>
<td>Shotcrete panel deflection</td>
</tr>
<tr>
<td>$\sigma_{sa}$</td>
<td>Shotcrete adhesive bond strength, MPa</td>
</tr>
<tr>
<td>$\sigma_s$</td>
<td>Shear strength</td>
</tr>
<tr>
<td>$\sigma_t$</td>
<td>Tensile strength (MPa)</td>
</tr>
<tr>
<td>s</td>
<td>Block width (m)</td>
</tr>
<tr>
<td>SIMRAC</td>
<td>Safety in Mines Research and Advisory Committee</td>
</tr>
<tr>
<td>RYHP</td>
<td>Rapid Yielding Hydraulic Props</td>
</tr>
<tr>
<td>$T_c$</td>
<td>Direct shear capacity (tau), MPa</td>
</tr>
<tr>
<td>$T_d$</td>
<td>Direct shear demand (tau), MPa</td>
</tr>
<tr>
<td>t</td>
<td>Thickness (mm)</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
</tr>
<tr>
<td>--------------</td>
<td>--------------------------------------------------</td>
</tr>
<tr>
<td>TSL</td>
<td>Thin Sprayed Liner</td>
</tr>
<tr>
<td>vs</td>
<td>versus</td>
</tr>
<tr>
<td>W</td>
<td>Weight (kN)</td>
</tr>
<tr>
<td>( w_b )</td>
<td>Bond width (mm)</td>
</tr>
<tr>
<td>( W_{pc} )</td>
<td>Peak load capacity</td>
</tr>
<tr>
<td>za</td>
<td>Shotcrete adhesive bond length</td>
</tr>
</tbody>
</table>

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

Safe and profitable mining is of paramount importance to underground mines in South Africa. In achieving these objectives, 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.

Methodologies for the design and selection of permanent areal support remain largely subjective. Testing methods for shotcrete, thin sprayed liners (TSL), mesh-type support and combinations of mesh, tendons and liners are reasonably varied and depend on the loading and boundary conditions of the test arrangement. The selection of temporary areal support systems are, similarly, subjective.

Recognising the need for a more structured approach to selecting areal support for particular mining environments, the Mine Health and Safety Council (MHSC), through the Safety in Mines Research and Advisory Committee (SIMRAC), initiated a research project to address the topic. The project was carried out as Safety in Mines (SIM) 15 02 02 and aimed to design a feasible methodology for selecting permanent areal support in varying underground mining environments, namely tabular, conventional and semi-mechanised (bord and pillar), gold and platinum mining operations in South Africa. The project relied on support installation methodologies that are currently being practiced or that have been trialled.

This booklet contains the findings from the research project – namely a methodology to guide the selection of areal support. While the original project was directed specifically to the selection of permanent areal support, the adaptability of the methodology allows the user to apply the systematic process to the selection of temporary areal support, provided that appropriate tests results are available. It is intended that the methodology serve as a guideline, and thus is not prescriptive. An areal support ranking spreadsheet tool has been developed according to this methodology.
2. 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 2-1) 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 2-1.

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 3.
Table 2-1: Support systems evaluated at participating mines

<table>
<thead>
<tr>
<th>Support system number</th>
<th>Support system description</th>
<th>Participating mines</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Steel rope mesh (netting) with tendons [high stope width]</td>
<td>Bambanani East Mine, Harmony Gold, Welkom</td>
</tr>
<tr>
<td>2</td>
<td>Steel rope mesh (netting) no tendons [low stope width]</td>
<td>Bambanani East Mine, Harmony Gold, Welkom</td>
</tr>
<tr>
<td>3</td>
<td>Shotcrete</td>
<td>Booysendal Mine, Northam Platinum</td>
</tr>
<tr>
<td>4</td>
<td>Shotcrete</td>
<td>Booysendal Mine, Northam Platinum</td>
</tr>
<tr>
<td>5</td>
<td>Mesh</td>
<td>Ikamva shaft (Sibanye Gold Mines’ Kloof 4 Shaft) and at Tau Tona, AngloGold Ashanti</td>
</tr>
<tr>
<td>6</td>
<td>Weld mesh</td>
<td>Tau Tona, AngloGold Ashanti</td>
</tr>
<tr>
<td>7</td>
<td>TSL</td>
<td>Two Rivers Platinum (ARM-Impala JV) bord and pillar operation</td>
</tr>
<tr>
<td>8</td>
<td>TSL</td>
<td>Tumela Mine, Anglo Platinum</td>
</tr>
<tr>
<td>9</td>
<td>Chainlink mesh</td>
<td>Lonmin plc Karee 4 Belt and Dishaba Mine (Anglo Platinum) (both are conventional narrow tabular platinum operations)</td>
</tr>
<tr>
<td>10</td>
<td>Weld mesh</td>
<td>Dishaba Mine, conventional stoping</td>
</tr>
</tbody>
</table>
Figure 2-1: Overview of the methodology for the selection of permanent areal support
2.1 Mining environment

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 2-2. 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 2-2: Aspects of the mining environment

<table>
<thead>
<tr>
<th>Aspect</th>
<th>Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mining depth</td>
<td>Deep ($\geq 1500$ m dbs); Intermediate ($500 \leq \text{dbs} \leq 1500$ m) and Shallow ($\leq 500$ m).</td>
</tr>
<tr>
<td>Stress</td>
<td>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)</td>
</tr>
<tr>
<td>Structure</td>
<td>Jointed vs unjointed rock masses (kinematic-driven wedge failures vs beam (shear) or stress-driven failures)</td>
</tr>
<tr>
<td>Location in working area</td>
<td>Mining face vs back area</td>
</tr>
<tr>
<td>Safety (through exposure of personnel)</td>
<td>Number of workers, duration of exposure and intensity of manual labour</td>
</tr>
<tr>
<td>Extent of mechanisation</td>
<td>Conventional vs semi-mechanised (bord and pillar) mining, manual vs mechanised</td>
</tr>
<tr>
<td>Accessibility</td>
<td>Low reach vs high reach excavations</td>
</tr>
<tr>
<td>Stoping width</td>
<td>Low stoping widths vs high stoping widths</td>
</tr>
<tr>
<td>Corrosiveness</td>
<td>Dry vs water conditions, low vs high PH, low vs high sulphate content</td>
</tr>
</tbody>
</table>
2.3 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 2-2. The capacity of areal support systems should be compared by considering these characteristics.

![Figure 2-2: Key support characteristics](image)

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.

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.

2.1.1 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 Appendix B.3.

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 Appendix B.1. TSL material characteristics are described in Appendix B.2.

2.1.2 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 Appendix E.
2.1.3. Shotcrete and TSL index tests

Yilmaz (2011) developed two new laboratory testing methods for the determination of TSL mechanical properties. The 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 are shown in Appendix D.

2.1.4. Energy absorption and yield testing for mesh and shotcrete

Ortlepp & Stacey (1997) (see Appendix E) 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), but it was necessary to assume corrections for the boundary conditions. The comparative results are presented in Figure 2-3. This graph is useful for comparative purposes.

2.4 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 is 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.
Different types of support will be more resistant to these effects.

![Diagram showing energy absorption and displacement for different support elements.]

**Figure 2-3**: 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)

### 2.5 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 detect 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 to install 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.

2.6 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.
3. 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 2, is presented in Table 3-2. 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 2.1) plays a significant role in the allocation of ratings.

3.1. Support capacity rating

The support specifications and data from support testing (section 2.3) are presented with green shading in the ranking tool (Table 3-2). Where the ranking tool support units are dissimilar to those tested, several corrections and/or assumptions have been applied (which are detailed in Appendix C) and presented with blue shading.

Initial stiffness, peak load and yield capacity ratings, ranging from 1 to 10, are then assigned on a comparative basis based on the support specifications and data.

The requirements of the support system to be installed depend on the mining environment. The 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 3-1. 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 3-1: Suggested weighting factors for mining environments

<table>
<thead>
<tr>
<th>Support system capacity</th>
<th>High stress and dynamic</th>
<th>Low stress (jointed) environment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial stiffness</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Yield capacity</td>
<td>0.4</td>
<td>0</td>
</tr>
<tr>
<td>Load capacity</td>
<td>0.3</td>
<td>0.7</td>
</tr>
</tbody>
</table>
The support system capacity is thus calculated using the equation:

\[ SSC_{\text{rating}} = IS_{\text{MEF}} \times IS_{\text{rating}} + YC_{\text{MEF}} \times YC_{\text{rating}} + LC_{\text{MEF}} \times LC_{\text{rating}} \]  
(Equation 1.)

Where

- \( IS_{\text{MEF}} \) = Initial stiffness mining environment factor
- \( IS_{\text{rating}} \) = Initial stiffness rating
- \( YC_{\text{MEF}} \) = Yield capacity mining environment factor
- \( YC_{\text{rating}} \) = Yield capacity rating
- \( LC_{\text{MEF}} \) = Load capacity mining environment factor
- \( LC_{\text{rating}} \) = Load capacity rating

### 3.2. 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:

\[ P_{\text{rating}} = \min(IQ_{\text{rating}}, BR_{\text{rating}}, ER_{\text{rating}}, CR_{\text{rating}}) \]  
(Equation 2.)

Where

- \( IQ_{\text{rating}} \) = Installation quality rating
- \( BR_{\text{rating}} \) = Blast performance rating
- \( ER_{\text{rating}} \) = Equipment performance rating
- \( CR_{\text{rating}} \) = 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 (RF). The performance factor is decile expression of the performance rating, \( PF = \frac{P_{\text{rating}}}{10} \).

The overall system performance is calculated using the equation:

\[ OP_{\text{rating}} = PF \times SSC_{\text{rating}} \]  
(Equation 3.)
3.3. Practicality rating

The overall practicality is a weighted rating of the overall handling, labour requirement and installation.

The overall practicality rating is calculated using the equation:

\[ P_{rating} = w_{OH} \times OH_{rating} + w_{LR} \times LR_{rating} + w_{I} \times I_{rating} \]  

(Equation 4.)

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

3.4. 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:

\[ OSS_{rating} = w_{op} \times OP_{rating} + w_{p} \times P_{rating} \]  

(Equation 5.)

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

3.5. Installed Cost

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

\[ \text{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 m²
Labour cost is determined as follows:
\[ \text{Labour cost (R/m}^2) = \frac{\text{Labour cost R/hr}}{\text{Installation time m}^2/\text{hr}} \]

\[ \text{Labour cost R/hr} = \frac{\text{cost per person (R/month) } \times \text{people involved}}{\text{hours worked per month (hr/month)}} \]
Table 3-2: Ranking tool for the selection of permanent areal support in underground mining environments

<table>
<thead>
<tr>
<th>Mining Environment</th>
<th>Case Study 1</th>
<th>Case Study 2</th>
<th>Case Study 3</th>
<th>Case Study 4</th>
<th>Case Study 5</th>
<th>Case Study 6</th>
<th>Case Study 7</th>
<th>Case Study 8</th>
<th>Case Study 9</th>
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<td>1.5</td>
<td>2.5</td>
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<td>26</td>
<td>36</td>
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<td>36</td>
<td>26</td>
<td>36</td>
<td>390</td>
<td>140</td>
</tr>
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</table>

**Legend**

Not applicable
Test result
Adjusted results
Specifications
Rating
### Table 3-3: Description and ratings for ‘effectiveness of support’

<table>
<thead>
<tr>
<th>Class</th>
<th>Description of classification</th>
<th>Rating</th>
</tr>
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<tbody>
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<td></td>
<td><strong>Blast performance</strong></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Heavily damaged cannot be fixed, new support requires to be installed</td>
<td>0 - 2.5</td>
</tr>
<tr>
<td>B</td>
<td>Moderate damage, fixed by rehabilitation</td>
<td>2.5 - 7.5</td>
</tr>
<tr>
<td>C</td>
<td>Little damage that does not need rehabilitation</td>
<td>7.5 - 10</td>
</tr>
<tr>
<td>D</td>
<td>No damage at all</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td><strong>Equipment performance</strong></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Heavily damaged cannot be fixed, new support requires to be installed</td>
<td>0 - 2.5</td>
</tr>
<tr>
<td>B</td>
<td>Moderate damage, fixed by rehabilitation</td>
<td>2.5 - 7.5</td>
</tr>
<tr>
<td>C</td>
<td>Little damage that does not need rehabilitation</td>
<td>7.5 - 10</td>
</tr>
<tr>
<td>D</td>
<td>No damage at all</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td><strong>Corrosive performance</strong></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Completely corroded</td>
<td>0 - 2.5</td>
</tr>
<tr>
<td>B</td>
<td>Severe damage</td>
<td>2.5 - 7.5</td>
</tr>
<tr>
<td>C</td>
<td>Superficial rusting</td>
<td>7.5 - 10</td>
</tr>
<tr>
<td>D</td>
<td>No visible rusting</td>
<td>10</td>
</tr>
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</table>
Table 3-4: Descriptions and ratings for ‘practicality’

<table>
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<th>Class</th>
<th>Description of classification</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Labour requirements</strong></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Highly labour intensive and manual installation</td>
<td>0 - 2.5</td>
</tr>
<tr>
<td>B</td>
<td>High labour intensity and semi -mechanised installation</td>
<td>2.5 - 5</td>
</tr>
<tr>
<td>C</td>
<td>Low labour intensity and semi -mechanised installation</td>
<td>5 - 7.5</td>
</tr>
<tr>
<td>D</td>
<td>Low labour intensity, highly mechanised installation</td>
<td>7.5 - 10</td>
</tr>
<tr>
<td></td>
<td><strong>Installation</strong></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Extremely difficult to apply or install</td>
<td>0 - 2.5</td>
</tr>
<tr>
<td>B</td>
<td>Very difficult to apply or install</td>
<td>2.5 - 5</td>
</tr>
<tr>
<td>C</td>
<td>Fairly difficult to apply or install</td>
<td>5 - 7.5</td>
</tr>
<tr>
<td>D</td>
<td>Easy to apply or install</td>
<td>7.5 - 10</td>
</tr>
<tr>
<td></td>
<td><strong>Materials handling</strong></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Very difficult to transport to the working face</td>
<td>0 - 2.5</td>
</tr>
<tr>
<td>B</td>
<td>Difficult to transport to working face</td>
<td>2.5 - 5</td>
</tr>
<tr>
<td>C</td>
<td>Fairly difficult to transport to the working face</td>
<td>5 - 7.5</td>
</tr>
<tr>
<td>D</td>
<td>Easy to transport to the working face</td>
<td>7.5 - 10</td>
</tr>
<tr>
<td></td>
<td><strong>Equipment handling</strong></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Very difficult to transport to the working face</td>
<td>0 - 2.5</td>
</tr>
<tr>
<td>B</td>
<td>Difficult to transport to working face</td>
<td>2.5 - 5</td>
</tr>
<tr>
<td>C</td>
<td>Fairly difficult to transport to the working face</td>
<td>5 - 7.5</td>
</tr>
<tr>
<td>D</td>
<td>Easy to transport to the working face</td>
<td>7.5 - 10</td>
</tr>
</tbody>
</table>
4. CONCLUSIONS

Detailed evaluations of ten permanent areal support systems in different mining environments were carried out including comprehensive photographic records, taking the following into consideration:

- Mining environment
  - Geology and geotechnical characteristics
  - Stress regime
  - Stoping width
  - Mining method
  - Mechanisation
- Support specifications
- Support performance
- Support installation
- Support and labour costs

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 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 temporary and permanent areal support systems. The mining environment plays a major role in the support performance and practicality.

The ten support systems were ranked according to this methodology and the results are presented in a table. In many cases, the support capacity is penalised due to performance factors. Scraper damage plays a major role in narrow stopes and in gullies that are full of broken rock. The method of attaching the areal support can
severely impact the quality of installation and overall performance. In conventional mines, the transport and installation of permanent areal support is often a challenge. In these mining environments, temporary nets may provide a more appropriate solution. In the ten permanent areal support systems evaluated, only two are installed routinely at the stope face, one was installed routinely in stope gullies and the remainder are ad hoc installations. This represents current practice.
5. REFERENCES


Fernandez-Delgado, G; Cording, E,J; Mahar, J, W; Jan, Van Sint; M, L (1981). Thin shotcrete linings in loosening rock., Easton, Maryland, USA: American Concrete Institute.


6. ACKNOWLEDGEMENTS

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- The Mine Health and Safety Council, for sponsoring the research;
- The rock engineering personnel at the participating project mines, namely:
  - Harmony Gold Mines (Bambanani)
  - Northam Platinum Mines (Booysendal)
  - Sibanye Gold Mines (Kloof 4 Shaft)
  - AngloGold Ashanti (Tau Tona)
  - ARM – Impala Platinum Mines (Two Rivers Platinum)
  - Anglo Platinum Mines (Tumela Mine and Dishaba Mine); and
  - Lonmin Platinum Mines (Karee 3 Shaft)
- The academic personnel at the University of Pretoria, for the use of their Virtual Reality Centre during the project initiation workshop; and
- Geobrugg Southern Africa (Pty) Ltd for facilitating site visits as well as providing technical product information.
7. LIST OF APPENDICES

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Appendix A - 3  In-stope shotcrete UG 2 stoping (low stope width < 2.2 m)
Appendix A - 4  In-stope shotcrete Merensky decline (high stope width > 3 m)
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Appendix A - 6  In-stope steel welded mesh
Appendix A - 7  In-stope steel welded mesh
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Appendix A

Overview of permanent areal support systems being currently implemented or trailed in South African tabular gold and platinum underground mines
Appendix A-1

Steel rope 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
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 as 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 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 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 scrapper 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 difficult 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 necessary for consideration in relation to the mine design objectives for profitability. Although there is a moderate cost associated with this support methodology (≈ R300 per m² 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 performance 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 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 wieldy 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 subsequently 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 researchers 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
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 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 same 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 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 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 scrapper 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 ($\leq 1.5 \text{ m}$) 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

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
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 ensure 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. Rating of 8 and 6 were subsequently assigned to equipment and materials handling. The application of shotcrete is semi mechanised and requires very few people resulting in labour requirements 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 response. 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 the primary support, and where borehole cameras 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 or 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 ideal form of areal support in mechanised environment because mobile machinery cannot pull and rip it apart like mesh

Limitations of shotcrete support:

- Since it was a 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 location where it will be applied, thereby presenting logistic issues
- It obscures the visibility of discontinuities and other geological features as well of rock bolts, making it impossible to see time deterioration.

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
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 stoping environment; except that this was in a development decline tunnel so the stoping 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 tunnel face. The design methodology was not specifically different from the UG 2 and the same shotcrete thickness of 50 mm was applied.

The same 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 ensure 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. Rating of 8 and 5 were subsequently assigned to equipment and materials handling. It should be noted that in the decline larger amounts of raw materials are handled as 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 relative larger labour complement in the decline as compared to the stopes, therefore labour requirements are rated 8. The installation of shotcrete is fairly easy and was rated as 8.
Appendix A-5

Steel welded mesh

Deep level mining in high stress, seismically susceptible, tabular hard rock environment using conventional (manual) stoping methods
A-5. Steel welded mesh

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, fairly good yield (deformation) capacity although it is 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 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
The 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 both 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 Hydrabolts holes and small water 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 Hydrabolts are transported from surface in rail bound material cars upto 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, consequently 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 their own tests and subsequently provide 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 wieldy 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 installation
Appendix A-6

Steel welded mesh

Deep level mining in high stress, seismically susceptible, tabular hard rock environment using conventional (manual) stoping methods
A-6. Steel welded mesh

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 pretty well with the observed ultra – strength chain link mesh.

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 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, fairly good yield (deformation) capacity although it is lower than chain – link mesh.

The mesh used at Tau Tona has larger strand diameters than the one 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, the transportation of which is much more difficult than the mesh at Ikamva due to the increased weight per unit area and mesh is stiffer (more difficult to install) therefore it was therefore 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 their own tests and subsequently provide 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 wieldy 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-16 to Figure A-18.
Figure A-16: Good weld mesh condition in a gully and split sets

Figure A-17: Weld mesh damage caused by machinery

Figure A-18: Weld mesh damage caused by blasting
Appendix A-7

Steel welded mesh

Shallow to intermediate level mining in jointed and static, tabular hard rock environment using conventional (manual) stoping methods
A-7. Steel welded mesh

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 tandem with the installation procedures and standards. This resulted in not such 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 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 it 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 their own tests and subsequently provide 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 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 wieldy 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-19 to Figure A-21.
Figure A-19: Blast damaged mesh still retaining rock

Figure A-20: Equipment 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
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 hanging wall 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 found supporting rock that had unravelled from the hanging indicating good quality installation, as a result installation quality was rated 9.

The mesh was installed right upto the mining face and was blasted on during observations no indications of blast damage where found however; the mesh was moderately damaged by equipment and 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 together in conjunction with mechanical end anchors. These are installed by making use of airlegs. The transportation of airlegs in the stopes is very easy 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 requirements rating of 5 was assigned. The mesh is fairly easy to install due to its ability to be rolled in one direction and thus ensuring it is installed correctly. Adjacent mesh panels are connected together by use of clips which is relatively easy to clip 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 where 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 experimental
- 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 as 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 of safety net installation, 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-23: Machinery damage on chain link

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

Figure A-25: Blasted on to 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
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, which give 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 has been 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. 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, TSLs 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 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 the rock mass looked generally better after upon 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 an utility vehicle or LHDs. However large amounts of raw materials need to be transported to the working faces, although there are relatively less than those of shotcrete. Rating of 8 and 6 were subsequently 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 seems 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 is produces is less 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 the required shotcrete thickness. This means that less bulky 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 procedure.
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.
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 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. 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 the ones 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 systems characteristics however due to the relatively similar behaviour between TSLs and shotcrete, TSLs 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 in the same ball park 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 where the thickness was less than the required thickness, it was marked off for respraying. Generally the rock mass looked generally better after upon application and interactions with the workers confirmed this sentiment whereby 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 the 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. Rating of 5 and 4 were subsequently 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 thorough as would be expected for mesh
- The TSL machine was relatively small and could be easily transported by 2 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 bound 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, subsequently the barred rock and the bonded TSL are detached from the hanging wall 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 TSLs.

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 barring.

Figure A-32: Insufficient TSL thickness application
Appendix B

Overview of design methodologies for material selection (shotcrete, TSLs and mesh)
B-1. Shotcrete

B-1.1 Shotcrete design methodology

The design methodology of shotcrete for application in underground mines was developed in a previous MHSC research project, namely SIMRAC (SIM 04 02 04). 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 B-1.

The design of shotcrete is influence 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.

B-1.2. Shotcrete failure modes

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

- Adhesive failure,
- Direct Shear failure,
- Compressive failure,
- Flexural failure,
- Punching shear failure and
- Tensile failure

Summaries of the descriptions of the failure modes are given in the subsections that follow.
Figure B-1: Flow chart for shotcrete design in underground mines (modified after Joughin et al., 2012)
Figure B-2: Shotcrete failure modes (Barrett and McCreath, 1995).
Adhesive failure

This occurs when there has been a loss of adhesive bond between the shotcrete and the rock surface. 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 must fail in direct shear.

Adhesion Demand

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

\[ A_d = W = \frac{\rho g a b^2}{2\sqrt{3}} \]  

(Equation 6.)

Where

- \( a, b \) are the larger and smaller tendon spacings respectively,
- \( \rho \) is the density of the rock, and
- \( g \) is the gravitational acceleration (9.8 m/s²)

Adhesion Capacity

The capacity of a shotcrete lining to resist debonding (\( A_c \)) for a rectangular pattern is

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

(Equation 7.)

Where

- \( \sigma_{sa} \) 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).
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.

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:

$$T_d = W = \frac{\rho ga b^2}{2\sqrt{3}}$$  \hspace{1cm} (Equation 8.)

Shear Capacity

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

$$T_c = 2(a + b)\sigma_{ss} h$$  \hspace{1cm} (Equation 9.)

Where

- $h$ is the thickness of the shotcrete, and
- $\sigma_{ss}$ 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 $\sigma_{sc}$ is the compressive strength of the shotcrete.
**Flexural failure**

Once debonding 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 other 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 following equation for determination of the moment demand:

\[
m_g = \frac{\rho g a^2 b (3b - a)}{96 \sqrt[3]{3} (a + b)}
\]

(Equation 10)

Where

- \(a, b\) are the larger and smaller tendon spacings respectively;
- \(\rho\) is the density of the rock; and
- \(g\) is the gravitational acceleration (9.8 m/s\(^2\)).

And, assuming a square pattern:

\[
m_g = \frac{\rho g a^3}{96 \sqrt[3]{3}}
\]

(Equation 11)

**Moment capacity**

If shotcrete undergoes deformation, 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:
\[ \delta_d = 0.75 \frac{\delta_c}{b} \]  

(Equation 12.)

Where

\(\delta_c\) 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 B-3 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:

\[ m_c = \frac{h^2 W_{pc}}{0.0312} \]  

(Equation 13.)

Where

\(W_{pc}\) is the peak load (kN)

\(h\) is the thickness of the applied shotcrete
Figure B-3: Load deflection graphs for ASTM C1550 round panel tests

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

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.

During observations at the participating mining operations, data regarding geotechnical characterisation as well as areal support specifications were collected. At Booysendal Platinum (one of the champion mines) 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**

Booyendal encountered bad ground in its shallow bord and pillar mining operation. The bords are 8.0 m wide × 2.0 m high and is located in a blocky rock mass and areal support is required. The density of the rock is 3000 kg/m³ and the excavation is supported by 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 containment support will be required.
- Quasi-static **displacement** – not expected.
- The **weight** of the rock prism to be supported: \( W = 68 \text{ kN} \)

\[
A_d = W = \frac{\rho g ab^2}{2\sqrt{3}}
\]

\( \rho = 3000 \text{ kg/m}^3 \)
\( g = 9.8 \)
\( a = b = 2.0 \text{ m} \)

- Adhesion capacity for low bond strength \( A_c = 400 \text{ kN} \)

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

\( \sigma_{sa} = 1.0 \text{ MPa} \)
\( z_a = 50 \text{ mm} \) (the hanging wall surfaces were thoroughly cleaned with pressurized 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 \( T_d = W = 68 \text{ kN} \)
- Design shear strength: \( \sigma_{ss} = 3.4 \text{ MPa} \)
- Direct shear capacity \( T_c = 2(a + b)\sigma_{ss} h \)
- Required **thickness**: 25 mm,
- \( T_c = 2(2+2)*3.4*25 = 680 \text{ kN}, \text{ FoS} = 10 \)

So 25 mm thickness of shotcrete will ensures a FoS of 10.
B-2. Thin Spray-on Liners

B-2.1. TSL design methodology

B-2.1.a. Small deformations

Adhesive bond failure of TSL is assumed not to occur in small rock deformations. The expected failure modes are either direct shear or diagonal tensile rupture of the liner (Tannant, 2001). These expected failure modes at small deformations are shown in Figure B-4.

Figure B-4: Liner failure modes at small block displacements (Tannant, 2001), (a) direct shear and (b) diagonal tensile rupture

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 were discussed in the literature review. Equation 14 was used to calculate the support capacity of one of the liners from the champion mines for the research project, presented in Table B-1.
Table B-1: TSL specifications from Tumela mine (a ‘champion’ mine)

<table>
<thead>
<tr>
<th>Liner Specification</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Liner thickness (mm)</td>
<td>8.0</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>7.5</td>
</tr>
<tr>
<td>Tensile-bond (adhesive) strength (MPa)</td>
<td>2.5</td>
</tr>
<tr>
<td>Shear-bond strength (MPa)</td>
<td>6.5</td>
</tr>
<tr>
<td>Material Shear strength (MPa)</td>
<td>17.0</td>
</tr>
</tbody>
</table>

\[ F = t \cdot \sigma_t \]  

(Equation 14.)

Where

- \( t \) is the thickness of the TSL (mm)
- \( \sigma_t \) is the tensile strength of the TSL (MPa)

Using Equation 14, a tensile and shear support capacities of 60 kN/m and 136 kN/m respectively can be derived. Assuming a 1 m x 1 m bolt pattern, the maximum possible block size that can be detached is 1 m². Considering rock with a density of 3 000 kg/m³, 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. Underground loading conditions are irregular and this can reduce the TSL’s support capacity immensely.
**B-2.1.b. Large deformations**

Large pull-out tests done on 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. As a consequence, Tannant (2001) provides a failure mechanism for TSL under large deformations, that is, adhesion loss is followed by tensile rupture.

The force required to initiate adhesive debonding can be determined using the same logic as Equation 14. 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 15 which is less than the calculated tensile strength.

\[
F = \sigma_a \times w_b
\]  
*(Equation 15.)*

Where

- \(\sigma_a\) 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), a condition required for adhesive failure to occur (Tannant, 2001).

Since the liner's tensile strength is less 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 B-5.
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 16:

$$A = 4\sigma_a (s + 2x) w_b = W$$

(Equation 16.)

Where

- $W$ is the weight of the block
- $w_b$ is the bond width as before (Equation 15)
- $\sigma_a$ is the adhesive strength acting over the effective bond width
- $s$ 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 17:

$$T = 4s \cdot \sigma_t \cdot t$$  \hspace{1cm} (Equation 17.)

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 $\theta$ can be evaluated using Equation 18:

$$\theta = \arcsin \left( \frac{W}{T} \right)$$  \hspace{1cm} (Equation 18.)

The angle $\theta$ 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 19:

$$d = x \tan \theta$$  \hspace{1cm} (Equation 19.)

Based on the model presented in Figure B- 5 at the moment of tensile failure the following relationship holds (Equation 20):

$$\sigma_t \cdot s \cdot t \cdot \sin \theta = \sigma_a (s + 2w) w_b$$  \hspace{1cm} (Equation 20.)
B-3. Mesh

B-3.1. 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 B-6 which components 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 21:

\[
K_m = \frac{(L_d - L_{20})}{(D_d - D_{20})} \quad \text{(Equation 21.)}
\]

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, $D_0$: defined as the intersection of the line used to calculate the stiffness and the x-axis

In some cases, there are loads that are higher than the peak; however, beyond the design and peak loads the behaviour of the mesh is inconsistent (Dolinar, 2009). 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.

![Load - displacement curve for a test showing parameters used to evaluate mesh performance](image)

**Figure B-6:** 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 22:

\[ L_m = K_m \times D_m + L_o \]  

(Equation 22.)

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 B- 7.

![Figure B- 7: Linear load-displacement curves for different wire diameters (displacement measured in inches, load measured in pounds)](image-url)
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 is unknown, 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, 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. 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 velocity for ejected rocks.

\[ \text{Energy} = \frac{1}{2}mv^2 + mgh \]  
\[ \text{(Equation 23.)} \]

Where
m is the mass of the ejected rock
v is the velocity of the ejected rock
g is the acceleration due to gravity

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 path length element.

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 equal to the work done to the system. The energy capacity of the mesh is then compared to the demand which is determined as explained above.
Appendix C

Assumptions and corrections used alongside the ranking tool
Table C-1: Assumptions and corrections in the assignment of ranking values for support performance - Potvin Corrections

C-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

- Booyssendal applied 50 mm of unreinforced shotcrete which is similar to the shotcrete tested 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 C-2: Assumptions and corrections in the assignment of ranking values for support performance - Canadian Handbook for Rockbursts (CHR)

C-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:
   \[
   \text{Load increase factor} = \left(\frac{1770}{550}\right) \times \left(\frac{3}{3.8}\right)^2 = 2.0
   \]

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 \times displacement
   Energy increase factor = 2.0 \times k = 2.0 \times k
   where \(k > 1\)
Table C-3: Assumptions and corrections in the assignment of ranking values for support performance (Western Australian School of Mines (WASM) results)

C-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:
   \[
   \text{Load} = \text{Steel strength} \times \text{Strand area}, \text{with diameter being the only variable}
   \]
   \[
   \text{Load increase factor} = \left(\frac{4}{5.6}\right)^2 = 0.51
   \]

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
   \[
   \text{Energy} = \text{Force} \times \text{displacement}
   \]
   \[
   \text{Energy increase factor} = 0.51 \times 0.72 = 0.37
   \]

- 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:
   \[
   \text{Load} = \text{Steel strength} \times \text{Strand area}, \text{with diameter and steel strengths being the variables}
   \]
   \[
   \text{Load increase factor} = \left(\frac{1770}{550}\right) \times \left(\frac{3}{3.2}\right)^2 = 2.83
   \]

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) \[
\text{Energy} = \text{Force} \times \text{displacement}
\]
   \[
\text{Energy increase factor} = 2.83 \times k = 2.83k
\]
   where \(k > 1\)
Appendix D

Testing methods developed by Yilmaz (2011)
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 E-1. 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.

Shear movement on the rock-TSL boundary develops shear stress ($\tau_b$) which can be calculated from:

$$\tau_b = \frac{F}{\pi D t}$$

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 E-1: 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 let to cure. Superimposing 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 are shown in Figure E-2.

![Material shear strength apparatus](image)

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

The shear strength ($\sigma_s$) 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 i.e.;

$$\sigma_s = \frac{F}{\pi \times d_1 \times t}$$

Where;

- $d_1$ = steel-punch diameter in m
- $t$ = mean thickness of TSL in m

$$\sigma_s = \frac{2F}{\pi \times (d_1 + d_2) \times t}$$

Where

- $d_2$ = 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 E-3, 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.

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.

The calculation of tensile stress takes into account the original cross sectional area of the narrow section of the specimen. The following formula is used for calculating the tensile strength ($\sigma_t$):

$$\sigma_t = \frac{F}{A}$$

Where;
F = load at failure in N
A = original cross-sectional area of the specimen (in m$^2$) at the narrow section
Figure E- 3: Configuration of the tensile strength test assembly to be used in compressive machines (Yilmaz, 2011).
**Tensile-bond strength testing**

The Figure E- 4 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.

![Diagram of tensile-bond strength testing](image)

**Figure E- 4: Configuration of the specimen in tensile-bond strength testing**

(Yilmaz, 2011)

The fixture used in tensile-bond strength testing is shown in Figure E- 5. The cured specimen is placed on the bridging plate so that the rock substrate remains on the top as shown in Figure E- 5. 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.
The loading of TSL is done by load control method at a constant. 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.

![Configuration of the tensile-bond strength test assembly](image)

**Figure E- 5:** Configuration of the tensile-bond strength test assembly to be used in compressive machines (Yilmaz, 2011).

\[
\sigma_{tb} = \frac{F}{A} = \frac{F}{\pi \times r^2}
\]

Where

\( r \) = TSL radius
Appendix E

Support system capacity tests
E-1. 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.

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 E- 6: Dynamic test loading facility (Stacey & Ortlepp, 1999)

Figure E- 6 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.
E-2. 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 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.

E-2.1. Static test facility

Morton et al., (2008) have described the WASM static test facility and shown in Figure E-7. 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. The boundary restrain is shown in Figure E-8.

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 E-7: Static test facility

Figure E-8: Boundary restraint system
E-2.2. Dynamic test facility

Player et al., (2004) have described the WASM dynamic test facility shown in Figure E-9. 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 E-9: Dynamic test facility