Geotechnical Assessment of Thin Spray-on Liners for Underground Coal Mine Roof Support

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Geotechnical Assessment of Thin Spray-on Liners for Underground Coal Mine Roof Support

This thesis is presented in fulfilment of the requirements for the award of the degree of

Doctor of Philosophy

from

University of Wollongong

By

Zhenjun Shan

School of Civil, Mining and Environmental engineering

March 2017
CERTIFICATION

I, Zhenjun Shan, declare that this thesis, submitted in fulfilment of the requirements for the award of Doctor of Philosophy, in the School of Civil, Mining and Environmental Engineering, Faculty of Engineering and Information Sciences, University of Wollongong, is wholly my own work except where specific references or acknowledgements are made. The thesis has not been submitted for qualifications at any other academic institution.

Zhenjun Shan
March 2017
LIST OF PUBLICATIONS


ABSTRACT

Although thin spray-on liners (TSLs) have been stated as a potential medium for surface support in underground mines since the late 1980s, their application in underground coal mines is still in the infant stage. This thesis aims to evaluate the feasibility of replacing the traditional steel mesh with innovative TSLs for underground coal mine roof support. In this study, beam enhancement capacity of a TSL and its performance in supporting cracked rock mass were investigated; the TSL and steel mesh in supporting buckling strata, guttering strata and strata with weak bedding planes were compared; the load bearing capacities of TSLs, TSL-concrete composite and steel mesh were compared; numerical modelling was conducted to simulate the performance of the TSL in underground coal mine roof support.

Beam enhancement is one of the TSLs support mechanisms proposed by Stacey (2001), however, research on this mechanism has been limited. As such, four-point bending tests on plaster beams with and without TSL reinforcement were conducted to evaluate the ability of the TSL in enhancing roof beams. Test results indicated that the plaster beam experienced a significant increase in strength after being bonded with the TSL, failure modes of the beams were also altered. Previous studies on TSLs have mainly focused on determining their basic mechanical properties and load bearing capabilities. The mechanism by which TSLs help to stabilise the ‘cracked’ roof or walls of an underground roadway is not completely understood. To gain a better understanding, a series of hydrostone plaster beams with different notch shapes were tested to failure in a four-point bending test, as expected failure in all cases propagated from the tip of the notch. To assess the effect a thin spray-on liner (TSL) has in resisting crack propagation from the tip, another series of beams was coated on the notched face with a 5 mm thick polymeric liner. Results in all cases showed that failure at the crack tip had been resisted and that failure of the beam initiated elsewhere. This ability of the thin polymeric liner to resist crack propagation suggests that the polymer acts as a composite with the beam, a relationship which traditional steel mesh does not have. In order to investigate the influence of TSL material penetrating into rock joints, a series
of TSL reinforced notched beams with polymer filling in the notch were subjected to the four-point bending test. Results indicated that filling of the notch with polymer was able to change the failure mode of the beam.

Steel mesh has been successfully used in underground coal mines for a long time. However, as a passive support, it is not able to generate resisting force until substantial rock deformation occurs. Its inability of being installed automatically also has a detrimental influence in roadway advancement rate. Thin spray-on liners (TSLs) have been believed as a potential substitute for steel mesh in supporting underground coal mine roof for a long time, but research on comparison between the two types of surface support has been scarce. As such, an attempt to fill the gap in this research area was covered in this thesis.

Large scale laboratory tests were designed and conducted to compare the behaviours of a TSL and steel mesh which is currently used in Australian coal mines in supporting strata with weak bedding planes. It was found that while the peak load taken by the simulated rock mass with weak bedding planes sample with no skin confinement was 2494 kN, the corresponding value of the sample with 5 mm thick TSL reinforcement reached 2856 kN. The peak load of the steel mesh reinforced sample was only 2321 kN, but this was attributed to the fact that one of the rock bolts broke during the test. The TSL reinforced sample had a similar post-yield behaviour as the steel mesh reinforced one.

Another series of laboratory tests were designed and performed to compare TSLs and steel mesh in supporting buckling strata. Two types of TSL material were tested in this test, which included fibre reinforced polymer (FRP) B and FRP C. Results indicated that both of the two TSL materials reinforced samples had greater peak strength than the steel mesh confined samples. Compared with steel mesh, TSLs were able to provide stiffer support performance due to the intimate contact with the substrate surface. The stiff support performance is beneficial to roadway stability as it helps to
preserve the inherent strength of rock mass by restricting rock deformation. It was also found that the gap between steel mesh and the substrate not only reduced the peak strength of the sample but also decreased the stiffness of the sample.

The performances of TSL and steel mesh in reinforcing strata prone to guttering were also investigated by carrying out large scale guttering test in the laboratory. Results indicated that a TSL is better than steel mesh in restricting rock movement and thus inhibiting the formation of gutters in the roof.

One of the most important mechanical properties in assessing performance of surface support materials is their load bearing abilities. While previous investigations on determining the load bearing capabilities of steel mesh and TSLs were mainly based on small scale laboratory tests, full scale laboratory tests were developed and conducted in this study to better understand the in situ performance of the two surface support mediums. It was observed that while the plain TSL sheet was not as strong as the roof mesh which is currently used in the coal mines in Australia, the TSL-concrete composite was able to bear a slightly greater load than the roof mesh. Test results also showed that both the plain TSL sheet and the TSL-concrete composite provided much stiffer performance than the steel mesh, indicating that the TSL was superior in restraining rock deformation and thus maintaining the inherent strength of the rock mass.

In addition to the laboratory tests, three dimensional numerical modelling was also conducted to replicate the full scale laboratory test on steel mesh. The modelling was only conducted up to the first wire failure as input parameters after that point were not available. The modelling results matched well with the laboratory results. It was also indicated that the mesh subjected to loading by a domed platen would have similar results to the mesh loaded by a flat platen.
TSL is an innovative rock support medium, numerical studies on the performance of TSL in underground coal mine have been limited. The behaviour of a TSL in reinforcing the roof of an underground coal mine roadway was simulated. The results demonstrated that the TSL-rock bolt reinforced roof experienced significantly smaller displacement than the unreinforced roof and the rock bolt reinforced roof. In addition, the range of the rock deformation zone was also reduced in both the roof and rib. The sensitivity to mining depth and horizontal to vertical stress ratio was also studied. It was indicated that the TSL-rock bolt system was effective even at a mining depth of 800 m and horizontal to vertical stress ratio of 3 : 1.

The behaviour of a TSL in supporting roof strata was also studied using detailed numerical models. The results indicated that, compared with a rock bolt supported roof, a TSL-rock bolt supported roof had lower displacement and higher horizontal and vertical stresses, increasing the roof rock mass strength.
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# LIST OF SYMBOLS AND ABBREVIATIONS

## Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
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<tbody>
<tr>
<td>(\sigma_v)</td>
<td>Vertical gravitational stress (MPa)</td>
</tr>
<tr>
<td>(\gamma)</td>
<td>Unit weight of the rock (MN/m(^3))</td>
</tr>
<tr>
<td>(z)</td>
<td>Depth below ground surface (m)</td>
</tr>
<tr>
<td>(\sigma_h)</td>
<td>Gravity induced horizontal stress (MPa)</td>
</tr>
<tr>
<td>(v)</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>(d)</td>
<td>Rock sample diameter (mm)</td>
</tr>
<tr>
<td>(\sigma_a)</td>
<td>Axial stress (MPa)</td>
</tr>
<tr>
<td>(P)</td>
<td>Axial load (N)</td>
</tr>
<tr>
<td>(A)</td>
<td>Initial cross-sectional area of the sample (mm(^2))</td>
</tr>
<tr>
<td>(\sigma_{c=2})</td>
<td>Calculated UCS of the sample with height/diameter ratio of 2 (MPa)</td>
</tr>
<tr>
<td>(\sigma_{c&gt;1})</td>
<td>UCS of the sample with height/diameter ratio &gt; 1 (MPa)</td>
</tr>
<tr>
<td>(\sigma_{c#2})</td>
<td>UCS of the sample with height/diameter ratio not equal to 2 (MPa)</td>
</tr>
<tr>
<td>(h)</td>
<td>Height of the sample (mm)</td>
</tr>
<tr>
<td>(\Phi)</td>
<td>Internal angle of friction ((^\circ))</td>
</tr>
<tr>
<td>(C_{oh})</td>
<td>Cohesion (MPa)</td>
</tr>
<tr>
<td>(m)</td>
<td>Slope of the Coulomb strength envelope in terms of principal stresses</td>
</tr>
<tr>
<td>(x)</td>
<td>The value corresponding to the intercept of the strength envelope in axial stress axis (MPa)</td>
</tr>
<tr>
<td>(\sigma_t)</td>
<td>Tensile strength (MPa)</td>
</tr>
<tr>
<td>(P_f)</td>
<td>Failure load (N)</td>
</tr>
<tr>
<td>(t)</td>
<td>Thickness of the sample (mm)</td>
</tr>
<tr>
<td>(k)</td>
<td>Slope of the load versus displacement curve</td>
</tr>
<tr>
<td>(l_p)</td>
<td>Peak load (N)</td>
</tr>
</tbody>
</table>
$d_p$  Displacement at peak load (mm)

$l_{50}$  50% of the peak load (N)

$d_{50}$  Displacement at 50% of the peak load (mm)

$\sigma_f$  Flexural strength of the sample (MPa)

$l$  Span length (mm)

$b$  Sample width (mm)

$P_p$  Peak load (N)

$E_f$  Flexural modulus (MPa)

$F$  Force at a chosen point on the initial linear portion of the force-deflection curve (N)

$Y$  Deflection corresponding to force $F$ (mm)

$\sigma_{ad}$  Adhesion strength (MPa)

$A_i$  Area of the interface (mm²)

$\sigma_{sb}$  Shear bond strength (MPa)

$P_y$  Yield load (N)

$y$  Displacement at yield (mm)

$E$  Young’s modulus (MPa)

$I$  Second moment of area (mm⁴)

$k_n$  Normal coupling spring stiffness per unit area (N/m³)

$k_s$  Normal coupling spring stiffness per unit area (N/m³)

$K$  the bulk modulus (N/m³)

$G$  the shear modulus (N/m²)

$\Delta z_{min}$  the smallest dimension of an adjoining zone in the normal direction (m)

$\sigma_{NL}$  Normalised horizontal stress (MPa)

$E_N$  Normalised Young’s modulus (GPa)

xxvi
$E_M$  Measured Young’s modulus (GPa)
**Abbreviations**

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>TSL</td>
<td>Thin spray-on liner</td>
</tr>
<tr>
<td>TSLs</td>
<td>Thin spray-on liners</td>
</tr>
<tr>
<td>UPE</td>
<td>Unsaturated polyester</td>
</tr>
<tr>
<td>NVP</td>
<td>Non-vinyl pyrrolidone</td>
</tr>
<tr>
<td>FRP</td>
<td>Fibre reinforced polymer</td>
</tr>
<tr>
<td>MEKP</td>
<td>Methyl ethyl ketone peroxide</td>
</tr>
<tr>
<td>DMPT</td>
<td>Dimethyl p-toluidine</td>
</tr>
<tr>
<td>UCS</td>
<td>Uniaxial compressive strength</td>
</tr>
<tr>
<td>FR</td>
<td>Fibre reinforced</td>
</tr>
<tr>
<td>FLAC\textsuperscript{3D}</td>
<td>Fast Lagrangian Analysis of Continua in 3 Dimensions</td>
</tr>
</tbody>
</table>
INTRODUCTION

1.1 Background
The coal industry offers not only great economic but also social benefits to Australia. Australia has been a large energy exporter in the world for a long time (Australian Energy Update 2014). Among the various types of exported energy, coal plays an important role (shown in Table 1.1) with the share being over 60%. Coal is also a very important fuel for Australia, as it accounts for more than 30% of energy consumption and over 60% of electricity generated in Australia. To meet both the international and domestic requirements on coal production, the coal mining industry must operate in a safe and effective way.

Table 1.1 Coal share in Australia’s energy export, energy consumption and electricity generation

<table>
<thead>
<tr>
<th>Coal share in</th>
<th>2010-2011</th>
<th>2011-2012</th>
<th>2012-2013</th>
<th>2013-2014</th>
</tr>
</thead>
<tbody>
<tr>
<td>Energy export</td>
<td>60.5%</td>
<td>60.9%</td>
<td>61.2%</td>
<td>67.6%</td>
</tr>
<tr>
<td>Energy consumption</td>
<td>34.9%</td>
<td>34.2%</td>
<td>33.1%</td>
<td>31.7%</td>
</tr>
<tr>
<td>Electricity generation</td>
<td>68.2%</td>
<td>69.1%</td>
<td>63.9%</td>
<td>61.2%</td>
</tr>
</tbody>
</table>

Source: 2012-2015 Australian Energy Update

One of the most common safety issues in underground coal mines is roof instability. Unstable roof can induce roof falls which may result in injuries or even fatalities. Roof and rib falls account for approximately 50% of all fatalities in bituminous underground coal mines (Mark, Pappas & Barczak 2009). One currently used roof support system is bolts-and-mesh (shown in Figure 1.1). While rock bolts help to reinforce the rock mass, the mesh prevents loose rock materials from falling into the roadway. Steel mesh has been successfully used to control the roof strata in underground coal mines for a long time, however, it has many intrinsic disadvantages. Mesh is a passive support, and it does not generate support resistance until substantial displacement occurs (Tannant 1995). When large deformation takes place, the rock mass may have already
become loose or fractured, undermining the inherent strength of the rock mass and its self-supporting ability. It is common knowledge that it is difficult to hold the dead weight of the loosened rock mass. As such, preserving the inherent strength of the rock mass is essential to underground strata control (Brady & Brown 2006). Another disadvantage of the mesh is its inability to be installed automatically (Tannant 2004b). Its installation process is labour intensive and involves miners being exposed to unsupported roof, resulting in safety concerns as potential rock falls may occur during this period. The mesh installation is time consuming, which has an adverse effect on the roadway development rate. In addition, the environment in underground coal mines may result in mesh corrosion, reducing the load bearing capability of the mesh.

Figure 1.1 Bolt-and-mesh for underground coal mine roof support

As the currently used mesh has many shortcomings, an innovative roof support agent termed thin spray-on liner (TSL) was proposed to take the place of steel mesh for underground rock support. Current generation TSLs are polymer-based materials that generally fall into one of two material types: crosslinking polyurethane or polyurea based systems and cement-reinforced water-dispersible systems based on ethylene-vinyl acetate copolymer (Lukey et al. 2008; Espley-Boudreau 1999; Potvin et al. 2004).
TSLs have many superior properties over steel mesh as an alternative form of surface control in underground mines. As TSLs can to be automatically sprayed on the freshly excavated rock surface, the roadway advancement rate can be significantly increased so as to satisfy the requirement from the current modern high production mines (Nemcik et al. 2011).

The nature of TSLs enables them to have intimate contact with the rock surface. Unlike steel mesh, TSLs are able to produce resistance to rock movement at small displacements, thus the movement or dilation of the rock mass can be prevented at a relatively early stage. It is widely acknowledged that excessive strata movement is detrimental to roof stability. When significant displacement arises the fractured rock dilates along the cracks and therefore softens the rock mass, deteriorating the self-supporting ability of the rock. Unfortunately the current generation of TSLs do not achieve significant strength quickly enough to be used as immediate support for rapid roadway development in underground coal mines. As such, a new type of fast setting TSL, which used unsaturated polyester (UPE) and was crosslinked using non-vinyl pyrrolidone (NVP), was proposed and under development. This would allow rapid set and be suitable for use in roadway development in underground coal mines. The proof of concept TSL comprised a resin and glass fibre which were mixed and solidified to form fibre reinforced polymer (FRP). The resin was composed of two liquid components (part A and B). Both liquid components comprised an UPE dissolved in NVP. Part A included 2% of a cobalt naphthenate solution which acted as the catalyst. Part B included 2% of a methyl ethyl ketone peroxide (MEKP) solution acting as the promoter and 1% of dimethyl p-toluidine (DMPT) acting as the co-catalyst. The solidification (cure) process consisted of two steps which were gelation followed by vitrification. The material developed to be dimensionally stable in the gelation process and gained the full strength in the vitrification process. The proof of concept TSL was able to cure in seconds, enabling it to generate strength soon after application. As NVP is a potential carcinogen it was not suitable for further development for use underground. As such, prototype formulations were developed using various monomers. The testing program developed was used to support the development of program.
1.2 Problem statement

Even though the idea of using TSLs to provide surface support in underground mines was firstly proposed by Professor Archibald in the late 1980s (Potvin, Stacey & Hadjigeorgiou 2004), TSLs have still not been applied routinely in the mining industry. Yilmaz (2011) stated previously that the application of TSLs was in its infancy, it is still the case nowadays. Previous studies on TSLs were mainly on developing appropriate testing methods (Tannant et al. 1999; Lewis 2001; Saydam et al. 2003; Spearing et al. 2004; Ozturk & Tannant 2004; Yilmaz 2007; Nemcik et al. 2009; Yilmaz 2010; Qiao et al. 2014), determining the basic mechanical properties of TSLs (Spearing & Gelson 2002; Kuijpers et al. 2004; Ozturk 2005; Yilmaz 2011; Qiao et al. 2014) and evaluating the load bearing capacities of TSLs (Tannant 1997; Connor et al. 2003; Swan & Henderson 2004; Kuijpers 2004; Nemcik et al. 2009; Nemcik et al. 2011a), however, research on the other important aspects of TSLs, particularly in relation to coal measure rocks, has been limited. For example, the mechanism by which TSLs reinforces the rock strata has not been fully understood; cracks in the rock mass usually exist, the performance of TSLs in resisting crack propagation has not been studied; roof strata may contain weak bedding planes, the behaviour of TSLs in this condition has not been investigated; buckling and guttering may occur in the rock mass, the ability of TSLs in supporting rock in these situations has not been evaluated.

TSLs have long been considered as a potential alternative to steel mesh for underground rock support. Comprehensive studies have been conducted on the TSLs and steel mesh separately, however, direct comparison between TSLs and steel mesh for rock support has been scarce. In addition, most of the previous laboratory tests on assessing the load bearing capacities of TSLs and steel mesh were on a small scale, which may not accurately represent their in situ behaviour.

As mentioned above, there have been many gaps in the research on TSLs, this study is an attempt to address those gaps.
1.3 Objectives of the study
This study primarily aims to evaluate the feasibility of replacing the traditional steel mesh with innovative TSLs for underground coal mine roof support. In order to complete the primary goal the following objectives are covered in this study:

(i) Determine the basic mechanical properties such as tensile strength, flexural strength, adhesion strength and shear bond strength of TSLs.
(ii) Determine the mechanical properties of steel mesh.
(iii) Evaluate the beam enhancement capacity of a TSL and investigate its performance in supporting cracked rock mass.
(iv) Compare the TSL and steel mesh in supporting strata with weak bedding planes.
(v) Compare the TSL and steel mesh in supporting buckling strata.
(vi) Compare the TSL and steel mesh in supporting guttering strata.
(vii) Compare the load bearing capacities of TSLs, TSL-concrete composite and steel mesh using full scale laboratory testing.
(viii) Develop numerical models to match the behaviour of steel mesh, and develop a numerical model to simulate the performance of the TSL in underground coal mine roof support.

1.4 Outline of the thesis
This thesis is presented in 7 chapters. In the first chapter, the disadvantages of current surface control medium (steel mesh) in underground coal mines and the advantages of an innovative rock support material (TSL) are given. A prototype TSL, which cures in seconds and has the potential to replace steel mesh for underground coal mine roof support, developed in the University of Wollongong is described. The research gaps are highlighted and the primary objectives of this study are presented.

Chapter 2 reviews literature on three subjects related to this research. The first subject focus on the virgin stress in underground coal mines, failure modes of the roof of roadways and principles for roof support. In the second subject, the factors influencing
the behaviours of welded steel mesh in the laboratory tests are covered. In addition, the advantages and disadvantages of steel mesh for underground coal mine roof support are listed. The third subject describes thin spray-on liner (TSL) for roof support, which mainly includes TSL support mechanisms, advantages and disadvantages of TSL for roof support, previous tests to determine the basic mechanical properties and load bearing capacities of TSLs, numerical modelling of TSL in rock support and in situ trials of TSL in coal mines.

Comprehensive laboratory tests to determine the fundamental mechanical properties of sandstones, hydrostone plaster, TSLs and steel mesh are covered in Chapter 3. Factors influencing the mechanical properties of the hydrostone plaster were also investigated. Test results in this chapter helps to analyse the laboratory tests in the following chapters and also provides essential parameters and information for the numerical modelling conducted in this study.

In Chapter 4, the beam enhancement capacity of a TSL and its ability to reinforce cracked strata was investigated using a newly designed laboratory testing method. Laboratory tests were also developed to compare TSLs and steel mesh in supporting strata with weak bedding planes, buckling strata and guttering strata.

Chapter 5 evaluates the load bearing capacities of TSLs and steel mesh. Full scale laboratory testing was developed and then conducted on TSLs and two types of steel mesh. As the TSLs are bonded to the rock in the in situ application, a TSL-concrete composite sample was also prepared and subjected to the full scale test. The test results were compared to assess the feasibility of replacing steel mesh with TSLs for underground roof support.

In Chapter 6, numerical modelling was first conducted to match the three-point bending test on steel wire and then repeat the full scale laboratory test on steel mesh to get a better understanding of the behaviour of the steel mesh. In addition, the
behaviour of TSLs in underground coal mine roof support was also investigated using numerical modelling.

Chapter 7 summarizes the major results, achievements and conclusions of this thesis. In addition, recommendations for further work to extend this study are also included in this chapter.

1.5 Scope of work
In this study, laboratory tests were designed and conducted to compare the performance of TSLs and steel mesh in supporting rock mass with specific geological features (weak bedding planes, buckling and guttering), which was to help to evaluate the potential of TSLs in replacing steel mesh in underground coal mine roof support. Additionally, the load bearing capacities of steel mesh and TSLs were compared using full scale laboratory tests. There was no intention to make a comparison between TSLs and other traditional surface support mediums, such as shotcrete. As such, the advantages and disadvantages of shotcrete were not presented in this study, and there was no laboratory test conducted to investigate the performance of shotcrete.

The experiments conducted in the laboratory were all subject to static loading, dynamic loading was out of the scope of this study. In addition to laboratory tests, numerical modelling was also conducted in this study. The behaviour of full scale steel mesh sheets subject to pulling was simulated using FLAC3D. The performance of a TSL in combination with rock bolts in supporting the roof of a 10 m long underground coal mine roadway was investigated using numerical modelling. Also, the influence of mining depth and ratio of principal horizontal and vertical stress on the performance of the TSL was evaluated.
2 LITERATURE REVIEW

2.1 Introduction
Roof stability has always been a problem for underground coal mines. To successfully control the roof, it is necessary to understand roof support principles and the support mechanisms of the particular support system applied. Before any support technique is used underground, its potential to fulfil requirements should first be verified in the laboratory and then in situ.

Compared with steel mesh TSLs are a relatively new support technique under development. In this chapter, previous studies related to roof support principles, TSL support mechanisms, previous tests on TSLs and steel mesh, and comparison between TSLs and steel mesh are reviewed.

2.2 Roof failure modes and support principles
2.2.1 Virgin stress in underground coal mines
Underground rocks are in equilibrium state of stress before mining, this equilibrium is disturbed by mining. In order to design the optimal underground openings with appropriate geometry and orientation, it is important to evaluate the virgin stress state. There are several sources contributing to the virgin stress. They are gravitational stress, tectonic stress, thermal stress and residual stress, among which gravitational and tectonic stress are the most important ones.

Gravitational stress is the stress generated from the weight of the overburden strata. At a depth of ‘z’ below the ground surface, the vertical stress can be calculated using the following equation:

\[ \sigma_v = \gamma z \]  

Where:

\[ \gamma \]
σ_v = vertical stress (MPa)
γ = unit weight of the rock (MN/m³)
z = depth below ground surface (m)

As a result of the Poisson’s effect, the rock expands laterally due to the gravitational compression. The neighbouring rock strata has a constraining effect in response to the expansion, this produces the horizontal stress. The gravity induced horizontal stress is given by:

$$\sigma_h = \frac{\nu}{1-\nu} \sigma_v$$  \hspace{1cm} (2.2)

Where:

σ_h = horizontal stress (MPa)
ν = Poisson’s ratio

A typical rock Poisson’s ratio is 0.25 (Hoek 2001). Substituting this value into Equation (2.2) gives σ_h = σ_v/3, which suggests that the vertical stress is much greater than the horizontal stress, however, in situ measurements show that the horizontal stress is generally significantly greater than the vertical stress. This is due to the tectonic stress, which is generated by the lateral movement of continental plates forming the Earth’s crust. The relationship between horizontal stress and overburden depth in Australian coal mines is shown in Figure 2.1 where coloured symbols represent the maximum horizontal stress measured in different coal mines.

Figure 2.1 Increase of horizontal stress with depth in Australian coal mines as measured underground by SCT (Nemcik et al. 2006)
In underground coal mines, the stress state after excavation is the resultant of the pre-mining state of stress and mining induced stress. Determination of the initial stress state is a necessary part for roof support design. The stress state includes not only the stress magnitude but also the stress orientation. As shown in Figure 2.2, the stress orientation with respect to the roadway driven direction have significant influence on the roadway stability (SCT n.d.1).

The stress concentration level at the roadway face increases as the angle between roadway driven direction and major horizontal stress orientation grows. Specifically, when the roadway is driven parallel to the major horizontal stress, the roadway experiences minimal stress concentration and its stability is highest. On the other hand, the stress concentration reaches the highest level when the roadway is perpendicular to the major horizontal stress. In this situation, the full roadway width undergoes great stress concentration and the roadway stability is the least. Roadways that driven at an
acute angle to the major horizontal stress experience stress concentration at one corner of the roadway. When the stress is high enough, segments of the roof may fall out initially and guttering failure may develop eventually on that side of the roadway.

2.2.2 Roof failure modes

After mining, the stress equilibrium is disturbed and thus roof failure may occur. There are many roof failure modes resulting from various mechanisms. In order to effectively support the roof in underground coal mines, it is necessary to understand the roof failure modes. Roof failure modes have been studied by many researchers (Richmond et al. 1986; Su & Peng 1987; Bauer & Dolinar 2000; Dolinar et al. 2000; Van der Merwe et al. 2001; Zhang & Peng 2002; Gadde & Peng 2005a; Gadde & Peng 2005b; Shen 2014). In general, the mechanisms can be classified into seven types as listed below.

2.2.2.1 Span failure

Figure 2.3 describes the schematic of span failure. Span failure may occur when the roadway is too wide or the roof is not well supported. According to Richmond et al. (1986), there are two stages in span failure. In the first stage (Figure 2.3a), tensile cracks develop at the edges and the centre of the roadway as a result of the gravitational loading induced by the sagging strata. In the second stage (Figure 2.3b), the cracks develop further and eventually link together, leading to roof strata falls. It is important to note that while the cracks at both the edges and centre of the roadway are reported to be due to excessive tensile stress by Richmond et al. (1986), Singh and Ghose (2006) stated that the cracks at the roadway edge result from shear stress. In order to avoid this failure, it is necessary to reduce the sagging of the strata by increasing roof bolt density.
Figure 2.3 Span failure (Richmond et al. 1986)

2.2.2.2 Skin failure

As is shown in Figure 2.4, skin failure is characterised by the fall of small slabs of immediate roof material (Richmond et al. 1986). With respect to the size of the small slab, Richmond et al. (1986) defined it to be between 0.01 m$^2$ to 0.25 m$^2$ in area and Van der Merwe et al. (2001) described it as less than 0.3 m in thickness. Skin failure is a common failure type in underground coal mines. There are many factors contributing to this failure such as: low competence of the strata (friable or strongly bedded immediate roof, cross bedded strata units, slickensided strata units and geological discontinuities), roof being disturbed by continuous miners/mining practice, support not being installed in a timely manner, inadequate support density/excessive bolt spacing and weathering (Richmond et al. 1986; Bauer & Dolinar 2000; Van der Merwe et al. 2001; Zhang & Peng 2002). In order to control skin failure, many methods have been employed, including applying coating materials, rock bolts in
combination with wood headers, planks, oversized plates, wire mesh, chain link mesh, synthetic grid material, hoist rope and wood dowels (Bauer & Dolinar 2000).

2.2.2.3 Failure due to roof structures

Figure 2.5 is a schematic of failure resulting from roof structures. The structures of the roof can significantly influence the stability of the roadway by compromising the roof competence. According to Richmond et al. (1986), the structures which may result in roof failure consist of joints, slickensides or greasy backs, faults and dykes.

The existence of joints in the roof causes the roof strata to be discontinuous, which significantly reduces the strength of the roof in terms of the ability to resist bending, shear and tensile forces. The joint frequency, orientation and density have a significant influence on the roof failure manner. Slickensides or greasy backs are characterised by low cohesion. A roof with this type of structure allows the rock to dislodge from the main roof rapidly, which has a detrimental effect on the roadway stability. Both faults and dykes may decrease the competence of the surrounding strata and thus contribute to the instability of the roof.
2.2.2.4 Mid span shearing

Mid span shearing failure is shown schematically in Figure 2.6. This type of failure would occur when a weak bedded immediate roof is subject to high horizontal stress. The failure can propagate to the bolting horizon. Smaller bolt spacing and stiffer cross support help to control this failure mechanism (Richmond et al. 1986).

2.2.2.5 Shear failure

Shear failure is illustrated in Figure 2.7. This failure mode may occur when the whole bolted strata are low in strength. The strata may shear off along the two ribs and cave (Zhang & Peng 2002). This failure model is similar to the span failure mentioned above, however, they have different failure mechanisms. Span failure initiates with tensile cracks which are due to the gravitational loading induced by sagging of the roof strata. Shear failure develops from shear fractures along the ribs.
2.2.2.6 Buckling and beam failure

Figure 2.8 illustrates buckling failure. Thinly laminated immediate roof is susceptible to this failure model. This type of failure initiates with bed separation and then the lamination is compressed horizontally. Buckling failure is likely to occur at the middle of the roadway. As described by Zhang and Peng (2002), buckling failure involves only the roof losing its stability but not the fall of roof rock. Shen (2014), in reporting Dolinar et al.’s (2000) study, indicates that beam failure may occur with the fall of the laminated roof rock when the buckling failure is allowed to develop further.

2.2.2.7 Guttering failure

Roof guttering failure is also called cutter failure. It is not unusual in underground coal mines and has long been a concern for underground roadway stability. The schematic of this failure model is shown in Figure 2.9. Guttering failure is a progressive failure which initiates with shear cracks at the corners of the roadway and may propagate
further into the roof rock resulting in large roof falls (Richmond et al. 1986; Su & Peng 1987; Gadde & Peng 2005a). The main factors which control the gutter formation and propagation were also studied and are listed as: vertical stress, horizontal stress, relative stiffness between the coal and its immediate roof, large topographic relief, bed separation and gas pressure, geological anomalies, strength of the roof and the direction in which roadways are developed with respect to the direction of the in situ horizontal stress (Su & Peng 1987; Gadde & Peng 2005b). The effect of relative angle between roadway driven direction and major horizontal stress on guttering failure was discussed previously in Section 2.2.1.

![Figure 2.9 Guttering failure (Richmond et al. 1986)](image)

**Figure 2.9** Guttering failure (Richmond et al. 1986)

### 2.2.2.8 Cantilever and high arch failure

According to Richmond et al. (1986), when the guttering failure develops further the fractures propagate into higher strata, a cantilever forms in the roof (Figure 2.10). The cantilever at the guttered end may dislodge downward as it is not fully confined in the vertical direction, which in turn induce a compressive force on the hinge point at the other end of the cantilever beam. When the compression is great enough, compressive crack may initiate at the anchored end of the cantilever.
If there is no effective support to control the cantilever beam, the cracks can develop further into higher strata. Once the cracks interlink during the propagation process, a major roof fall may occur due to limited cohesion in the failed rock mass. This is the final stage of guttering failure; high arch failure (Richmond et al. 1986). Figure 2.11 is an illustration of this failure mode.

2.2.3 Roof support principles

The underground rock mass is in a state of stress equilibrium before excavation, stress redistributes after excavation and the rock may need to be supported to maintain stability. Steel mesh in conjunction with rock bolts and shotcrete are the two conventional rock support methods currently used in underground mines. TSLs are an
innovative surface support method under development. No matter which method is applied, there are certain principles to abide by so as to effectively control the strata. Nemcik et al. (2008) stated that the rock mass is relatively stiff before the disturbed period as a result of the confinement of compressive stress, however, rock softening, bulking and subsequent strata movement into the mine opening may occur if ground confinement does not exist. While it is not practical to stop the mining induced fractures from forming, it is possible to enhance roadway skin conditions by applying a support system at the roadway face at an early stage. Brady and Brown (2006) summarised several principles for effectively supporting rock, including: (1) keep the rock mass undisturbed to preserve the inherent strength of the rock mass surrounding the excavation; (2) ensure the support system is able to accommodate the varying underground strata environment; (3) the support system should have the ability to prevent the mechanical properties of the rock mass from deteriorating; (4) direct contact between the rock surface and the support system is significant; (5) once the support system has been installed, it should not be removed or disturbed; (6) the support system should be installed after excavation as soon as possible; (7) the support system should be capable of coping with the displacement of the excavation surface.

2.3 Welded steel mesh for roof support
One of the traditional surface support techniques currently used in underground coal mines is steel mesh which is applied in combination with rock bolts. Actually, steel mesh has been used successfully in underground coal mines to control the roof and the rib since the 1950’s (Morton et al. 2007). Unlike TSL which provides active support, steel mesh is a passive medium. The behaviour of welded steel mesh has been widely studied by field investigation (Villaescusa 2004), numerical modelling (Gadde, Rusnak & Honse 2006) and laboratory testing (Tannant 1995; Villaescusa 1999; Thompson 2004; Tannant 2004a; Morton et al. 2007; Player et al. 2008; Dolinar 2006, 2009; Morton, Thompson & Villaescusa 2009).
2.3.1 Factors influencing the behaviour of welded steel mesh

Determining the behaviour of welded steel mesh in the laboratory was a fundamental part of this study. According to the previous studies (Tannant 1995; Thompson 2004; Tannant 2004a; Dolinar 2006; Gadde, Rusnak & Honse 2006; Dolinar 2009), the factors that have significant effect on the performance of steel mesh are listed below:

(i) steel wire diameter;
(ii) bolt spacing;
(iii) size of the loading area;
(iv) loading plate orientation;
(v) bolt tension;
(vi) load surface;
(vii) bearing plate size;
(viii) size of test sheet.

2.3.1.1 Effect of steel wire diameter

Steel wire is the basic element of welded steel mesh. The influence of steel wire diameter on the response of steel mesh was investigated by Tannant (2004a). Pull tests were conducted on steel mesh composed of different wire diameters (5.2 mm, 4.1 mm and 2.9 mm). The results confirmed that the larger the steel wire diameter the stronger the steel mesh in terms of load bearing capacity. The peak loads achieved for the three types of mesh were 38.2 kN, 24.3 kN and 14.8 kN respectively. It was also observed that upon loading the initial stiffness was almost identical for the three types of mesh, however, the displacement at peak load increased with increasing diameter. A slightly different result was presented in the study of Dolinar (2009) in which both the stiffness and peak load were reported to increase as the wire diameter increased. Note that the stiffness in this study was calculated by the slope of a line from the peak load to a point at 20% of the peak load using the following equation: \[ K_s = \frac{(L_p - L_{20})}{(D_p - D_{20})}, \]
where, \( K_s \) is the screen stiffness, \( L_p \) is the peak load, \( L_{20} \) is the load at 20% of the peak load, \( D_p \) is the displacement at peak load and \( D_{20} \) is the displacement at 20% of the peak load, whereas the stiffness reported by Tannant (2004a) was from the peak load to 50% of the peak load.
2.3.1.2 Effect of bolt spacing

Steel mesh is pinned to the surface of the roadway in underground mines with rock bolts and plates. It is common knowledge that the bolt spacing has significant influence on the performance of the steel mesh. An attempt was made by Thompson (2004) using laboratory tests to study the effect of bolt spacing on mesh behaviour. Instead of using a solid plate which was commonly used in other mesh tests, a steel frame fabricated from four steel angle sections was used in this test to simulate a rigid slab or block of rock. A sheet of steel mesh was placed on the top of the steel frame and then bolted into a concrete floor. During the test, the load was applied by pulling the steel frame upwards. It was found from Table 2.1 that a smaller bolt spacing tended to have a stiffer load-displacement behaviour and greater peak load, due to the fact that the smaller spacing enabled the load to be transferred from the loading area to the load bearing plates more directly. Similar results were also found in a study by Dolinar (2009).

<table>
<thead>
<tr>
<th>Mesh configuration</th>
<th>Peak load (kN)</th>
<th>Displacement at peak load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MsBs1010Po075</td>
<td>35</td>
<td>125</td>
</tr>
<tr>
<td>MsBs1515Po075</td>
<td>23</td>
<td>340</td>
</tr>
<tr>
<td>MsBs1515Po105</td>
<td>34</td>
<td>170</td>
</tr>
<tr>
<td>MsBs2020Po105</td>
<td>24</td>
<td>300</td>
</tr>
<tr>
<td>MsBs1515Ps075</td>
<td>33</td>
<td>330</td>
</tr>
<tr>
<td>MsBs2020Ps075</td>
<td>17</td>
<td>400</td>
</tr>
</tbody>
</table>

The indications of the 13 alpha-numeric characters (MmBbxxyyPpsss) defining the mesh configurations are presented as follows: assuming there is a Cartesian coordinate system with two orthogonal axes x-axis and y-axis.

(i) Mm = mesh orientation – m = s(quare) / o(blique), where square indicates the wires of the mesh are parallel or perpendicular to the axes, oblique means the wires have a 45° or 135° angle with the axes.
(ii) $B_b = \text{bolt orientation} - b = s(\text{quare}) / o(\text{blique})$, where square indicates the edges of the bearing plates are parallel or perpendicular to the axes, oblique means the edges of the bearing plates have a 45° or 135° angle with the axes.

(iii) $xx = \text{bolt spacing in dm}$, which is the span between two adjacent bolts in x-axis direction.

(iv) $yy = \text{bolt spacing in dm}$, which is the span between two adjacent bolts in x-axis direction.

(v) $P_p = \text{plate orientation} - b = s(\text{quare}) / o(\text{blique})$, where square indicates the steel angle sections of the loading frame are parallel or perpendicular to the axes, oblique means the steel angle sections of the loading frame have a 45° or 135° angle with the axes.

(vi) $ss = \text{square plate side length in cm}$, which is the length of the steel angle section of the loading frame.

Figure 2.12 shows the schematic of typical mesh configurations.
2.3.1.3 Effect of size of the loading area

Thompson (2004) investigated the influence of the size of the loading area on mesh performance. Table 2.2 shows the test results. It was discovered that loading area size relative to the bolt spacing had significant influence on the load bearing capacity and stiffness of the mesh. Specifically, when the loading area was not greater than half of the bolt spacing, increasing the size of the loading area resulted in a stiffer mesh but did not affect its load bearing capacity. With the bolt spacing being 2 m × 2 m, the peak loads of the mesh subjected to loading sizes of 1.05 m × 1.05 m and 0.75 m × 0.75 m were both around 17 kN, however, the displacements at peak load of the mesh were approximately 330 mm and 400 mm respectively and the first mesh was slightly stiffer. This may be because larger loading area was restrained by more steel strands and thus needed greater load to achieve the same displacement as the relatively smaller loading area. When the loading area was larger than half of the bolt spacing both the peak load and stiffness increased with increasing loading area size. The increase in the peak load could be because the bigger loading area was more of a distributed load rather than a concentrated load as was the case with the small loading area. Specifically, while the peak loads achieved by MsBs1515Po075 and MsBs1515Po105 were 23 kN and 34 kN respectively the pressures applied on the two types of mesh were 41 KPa and 31 KPa respectively.

Table 2.2 Effect of size of the loading area on mesh performance (Thompson 2004)

<table>
<thead>
<tr>
<th>Mesh configuration</th>
<th>Peak load (kN)</th>
<th>Displacement at peak load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MsBs1515Po075</td>
<td>23</td>
<td>340</td>
</tr>
<tr>
<td>MsBs1515Po105</td>
<td>34</td>
<td>170</td>
</tr>
<tr>
<td>MsBs2020Po075</td>
<td>25</td>
<td>500</td>
</tr>
<tr>
<td>MsBs2020Po105</td>
<td>24</td>
<td>300</td>
</tr>
<tr>
<td>MsBs2020Ps075</td>
<td>17</td>
<td>400</td>
</tr>
<tr>
<td>MsBs2020Ps105</td>
<td>17</td>
<td>330</td>
</tr>
</tbody>
</table>

Note: the codes defining mesh configuration are the same as in Table 2.1
2.3.1.4 Effect of loading plate orientation

When the loading plate was square, it was postulated that the plate orientation with respect to the mesh may be an influencing factor on the mesh response. This effect was studied by Thompson (2004). Two types of plate orientation were investigated which included square and oblique. Test results are presented in Table 2.3. It was demonstrated that the effect of plate orientation on mesh performance was affected by bolt spacing and loading area size. When the bolt spacing was 1.5 m × 1.5 m and loading area was 0.75 m × 0.75 m, square orientation loading had greater peak load than oblique orientation loading. This result was partly supported by Nemcik et al. (2009) who found that the ultimate strength of steel mesh welds loaded at 45° to the wire strands is around 40% of the wire strand tensile strength. However, when the bolt spacing increased to 2 m × 2 m, the peak load of square loading was lower than oblique loading.

<table>
<thead>
<tr>
<th>Mesh configuration</th>
<th>Peak load (kN)</th>
<th>Displacement at peak load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MsBs1515Ps075</td>
<td>33</td>
<td>330</td>
</tr>
<tr>
<td>MsBs1515Po075</td>
<td>23</td>
<td>340</td>
</tr>
<tr>
<td>MsBs2020Ps075</td>
<td>17</td>
<td>400</td>
</tr>
<tr>
<td>MsBs2020Po075</td>
<td>25</td>
<td>500</td>
</tr>
<tr>
<td>MsBs2020Ps105</td>
<td>17</td>
<td>330</td>
</tr>
<tr>
<td>MsBs2020Po105</td>
<td>24</td>
<td>300</td>
</tr>
</tbody>
</table>

Note: the codes defining mesh configuration are the same as in Table 2.1

2.3.1.5 Effect of bolt tension

Slippage, which reduces the stiffness of the mesh, was noted during the pull test on welded steel mesh in previous studies (Tannant 1995; Thompson 2004). The slippage can be eliminated or at least reduced to some degree by applying a greater bolt tension. The effect of bolt tension on the behaviour of mesh was evaluated by Dolinar (2006) by conducting large scale pull tests on steel mesh with different bolt tension. The test
results (Table 2.4) indicated that greater bolt tension generally produced larger yield load, peak load and stiffness of the mesh, however, excessive bolt tension could result in premature failure of the wire due to the higher load concentrations in the section of the wires in contact with the bearing plates. Note that the stiffness in this study was calculated by the slope of a line from the peak load to a point at 25% of the peak load. Another study of Dolinar (2009) also showed that greater bolt tension generally produced larger peak load and stiffer initial load-displacement behaviour.

<table>
<thead>
<tr>
<th>Bolt tension (kN)</th>
<th>Yield load (kN)</th>
<th>Peak load (kN)</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25.6</td>
<td>4.6</td>
<td>14.6</td>
<td>26.0</td>
</tr>
<tr>
<td>38.4</td>
<td>8.2</td>
<td>15.8</td>
<td>35.4</td>
</tr>
<tr>
<td>51.2</td>
<td>8.0</td>
<td>9.3</td>
<td>36.7</td>
</tr>
</tbody>
</table>

2.3.1.6 Effect of load surface

The load surface is also one of the factors that determine the performance of steel mesh in the pull test. Both wood and steel plates were used for the pull tests by Dolinar (2006). It was found (in Table 2.5) that the load surface affected the behaviour of the mesh. Specifically, the yield load, peak load and stiffness increased when changing the load surface from steel to wood. The possible reason could be that the steel load surface induced a higher stress concentration in the wires than the wood load surface. In underground coal mines, steel mesh is loaded by rocks of varying stiffness, so the effect of variations to the bearing plate may be of more interest.

<table>
<thead>
<tr>
<th>Load surface</th>
<th>Yield load (kN)</th>
<th>Peak load (kN)</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>3.6</td>
<td>12.4</td>
<td>21.0</td>
</tr>
<tr>
<td>Wood</td>
<td>4.6</td>
<td>14.6</td>
<td>26.0</td>
</tr>
</tbody>
</table>
2.3.1.7 Effect of bearing plate size

Bearing plates are used in conjunction with bolts to fix the mesh during testing. The load is transferred to the bolts primarily by the wires directly under the bearing plates. The size of the load bearing plate directly influences load transfer as it determines the number of steel wires which lay directly under the bearing plates. Laboratory testing (Dolinar 2006) indicated that a bigger bearing plate produced greater peak load and stiffness of the mesh when the bolt tension was not excessive. This was due to the bearing plate having more steel wires passing underneath and thus the load can be distributed to more wires. It was also believed that larger plate involved less slippage than smaller plate. Note that the bearing plates in this test were square in shape. Similar results would be produced if circular bearing plates were used.

<table>
<thead>
<tr>
<th>Bearing plate size</th>
<th>Yield load (kN)</th>
<th>Peak load (kN)</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>152 mm × 152 mm</td>
<td>4.6</td>
<td>14.6</td>
<td>26.0</td>
</tr>
<tr>
<td>203 mm × 203 mm</td>
<td>3.0</td>
<td>24.6</td>
<td>51.1</td>
</tr>
</tbody>
</table>

2.3.1.8 Effect of size of the test sheet

Although not having an influence on actual mesh performance in situ the size of the test specimen does affect the results of the pull tests in the laboratory, but the data on the performance of full scale mesh is limited. Numerical modelling (Gadde, Rusnak & Honse 2006) was used to study the behaviour of mesh under full scale test conditions. It was found that the small sample size usually tested in the laboratory produced a higher maximum displacement than that gained in the full size mesh test as there was more restraint in the latter scenario. Therefore, a full size mesh, instead of small scale mesh, should be tested when accurate mesh behaviour is to be determined.
2.3.2 Advantages and disadvantages of steel mesh in roof support

Although steel mesh has been used successfully for a long time, it has some intrinsic advantages and disadvantages. This section is an attempt to summarise these advantages and disadvantages.

2.3.2.1 Advantages of steel mesh

The advantages of steel mesh are listed as follows:

(i) Standardised, there is no decision-making needed (Swan & Henderson 1999). Standards are existing with respect to mesh manufacture and installation procedures.

(ii) It is easy to control the quality (Swan & Henderson 1999). Flaws in mesh quality such as detachment at weld point and rusting can be easily observed. In order to better support the mine roof, steel mesh with excessive flaws should not be used.

(iii) It suits a wide variety of ground conditions and does not need special surface preparation (Swan & Henderson 1999). Steel mesh can be installed in both good and difficult strata condition. It does not involve blowing off the dust or excessive water content at roof surface as required by TSL before installation.

(iv) It offers good visual indications of ground behaviour (Swan & Henderson 1999). As is the nature of steel mesh, roof strata conditions, such as rock fractures and rock detachment, can be easily observed. As such, appropriate roof support measurements can be applied with respect to the special roof conditions during the mining process.

(v) Steel mesh is non-flammable and is able to conduct static electricity (Lukey et al. 2008). Static electricity generated in an underground coal mine may lead to fire and potentially an explosion, resulting in down time or injury to underground workers. Non-flammability is one of the most important requirements for coal mine roof support materials. As steel mesh is not flammable, roof support is less susceptible to damage when fire occurs.

(vi) The failure of steel mesh is plastic rather than catastrophic. This failure mechanism is desirable for underground coal mine roof support as it will
experience large deformation before it finally breaks. This allows the miners to have enough time to respond and helps to reduce roof fall injuries.

2.3.2.2 Disadvantages of steel mesh

The disadvantages of steel mesh are listed as follows:

(i) It is difficult to mechanise, apply and rehabilitate (Swan & Henderson 2004). Automation of the installation of steel mesh is hard to achieve, and once the steel mesh gets rusty, steel wires break or welds detach, it is difficult to rehabilitate.

(ii) Steel mesh is pinned onto the roadway roof as sheets using rock bolts, which limits the bolt placement pattern (Swan & Henderson 2004).

(iii) Vulnerable to blast damage (Hepworth & Lobato 2002).

(iv) Labour intensive and time consuming (Tannant 2004b). Unlike TSLs, the steel mesh cannot be automatically installed. Its application inevitably involves high level manual labour and is time consuming. High roadway development rate cannot be achieved in this case.

(v) Underground rock support installation is often involved in personnel injuries. Mesh installation requires manual labour (Tannant 2004b), which causes miners to be exposed to unsupported roof strata. Roof fall injury may occur at this stage.

(vi) Mesh is a passive support, it does not generate support resistance until substantial displacement occurs (Tannant 1995). One of the roof support principles is to keep the rock mass undisturbed to preserve the inherent strength of the rock mass surrounding the excavation, steel mesh support is not applicable to this rule.

(vii) Susceptibility to corrosion (Hepworth & Lobato 2002). Steel mesh is vulnerable to the humid environment in underground coal mines and will become rusty. The rusting reduces the load capacity of steel mesh, decreasing the roof support effectiveness of steel mesh.

(viii) Unable to prevent rock weathering (Hepworth & Lobato 2002). Unlike TSLs, steel mesh is not able to seal the roof rock to prevent it from weathering. Rock
weathering reduces the inherent strength of rock mass and makes ground control more difficult.

2.4 Thin spray-on liner for roof support

TSLs are an innovative surface support medium for underground strata control. They were first proposed by Professor Archibald of Queen’s University in Canada in the late 1980s (Potvin, Stacey & Hadjigeorgiou 2004) and have been attracting interest from both research institutes and industry since then. Due to the intrinsic advantages TSLs have been studied extensively. This section presents previous investigations on TSLs with respect to their support mechanisms, mechanical characteristics and field trials/application.

2.4.1 Support mechanisms of thin spray-on liner

Although mining induced fractures are impossible to prevent, it is important to control the fractured rock. For a support system, it is hard to bear the dead weight of rock once the rock mass has loosened (Hoek & Brown 1980). TSLs being a relatively new form of rock support in underground coal mines, their support mechanisms are not yet fully understood. It is common knowledge that TSLs, being an active support technique, are able to take action even if only small rock displacement occurs, which is desirable for rock support.

According to Tannant (2004b), although TSLs have a low load bearing capacity, they are able to inhibit rock mass dilation, loosening and unravelling in a fractured rock mass, establishing a stable zone as a result of making the rock elements interact with each other. If excessive dilation has already occurred, TSLs can work together with rock bolts to hold the loose rock. Additionally, a TSL’s displacement capacity is as important as their load bearing ability especially in cases where large displacement prevails (Tannant 2004b).
Stacey (2001) reviewed the support mechanisms of membranes. In that paper, membranes were defined as all ‘containment’ support elements, including wire mesh, shotcrete and TSLs, as opposed to retention support such as rock bolts. The containment mechanisms specific to TSLs are as follows:

(i) promotion of block interlock;
(ii) beam enhancement;
(iii) basket mechanism;
(iv) durability enhancement;
(v) extended ‘faceplate’.

2.4.1.1 Promotion of block interlock

Stacey (2001) stated that block interlock is beneficial to keep the rock mass stable and it can be illustrated as follows: (1) once the TSL is applied, the tension of the TSL and the bonding between the TSL and the rock can resist the shear on the interface so that block rotation is prevented (Figure 2.13); (2) mining induced cracks and joints often exist in the underground rock mass, TSL material can be sprayed into these joints and cracks in its liquid state and glue the rock mass together after it cures so as to improve block interlock (Figure 2.14). In reality, penetration of the TSL will not reach the tips of the cracks. A recent study (Fowkes, Teixeira de Freitas & Stacey 2008) found that even though the TSL cannot be sprayed to the crack tips, the TSL penetration is still able to help to redistribute the stress in the rock mass so as to decrease the stress intensity level at crack tips, which is helpful to prevent crack propagation and eventually benefit rock mass stability. The various sub-mechanisms also work together to restrain the dilation of the rock mass.
2.4.1.2 Beam enhancement

When the TSL is bonded to the roof beam, the roof beam stability may be improved as the enhancement is achieved in the bending performance of the roof beam (Stacey 2001).

2.4.1.3 Basket mechanism

This mechanism takes effect when rock failure or dilation occurs and then develops a basket in the membrane. In this case, the tensile strength of the membrane plays an important role in supporting the rock, and the flexural rigidity or membrane ductility affect the deflection of the membrane to produce a basket (Stacey 2001). A study by Tannant (2004b) has also proposed this mechanism.

2.4.1.4 Durability enhancement

Some rock types may deteriorate when subjected to wetting or drying. It is not unusual that the rock mass at the excavation degrades when it is exposed to air and moisture underground. In this situation, the application of membrane support is able to seal the
rock so as to prevent them from weathering and eventually preserve the inherent strength of the rock (Stacey 2001)

2.4.1.5 Extended ‘faceplate’

Like steel mesh, TSL is supposed to be applied together with rock bolts. According to Stacey (2001), the application of membrane will increase the area of influence of rockbolt and cablebolt faceplates (Figure 2.15). As the faceplate plays an important role in distributing the load, increasing the area of influence helps to transfer the load effectively.

![Extended faceplate action](image)

**Figure 2.15 Extended faceplate action (Stacey 2001)**

2.4.2 Advantages and disadvantages of thin spray-on liners in roof support

The use of TSLs in roof support has merit over steel mesh in spite of some minor shortcomings. This section presents the advantages and disadvantages of TSLs for strata control.

2.4.2.1 Advantages of thin spray-on liners

The advantages of TSLs are as follows:

(i) Rapid application rate makes it possible to install the support before the rock mass has time to loosen (Tannant 2004b). One of the roof support principles is
to install the support system as soon as possible after excavation (Brady & Brown 2006). The nature of a TSL enables it to be applied automatically soon after excavation, which helps to maintain the inherent strength of the rock.

(ii) In jointed or fractured rock mass, a TSL deters rock dilation, loosening and unravelling by promoting block interlock, creating a stable beam or arch of rock (Stacey 2001; Tannant 2004b).

(iii) Due to the intimate contact of TSLs with the rock surface, a TSL is able to generate support resistance at small rock deformations of millimetre order (Tannant 2004b). This support mechanism is desirable in underground strata control as it helps to maintain the inherent strength of the rock.

(iv) TSL application enables an increase in development rates and improves personnel safety (Nemcik et al. 2011). In contrast to steel mesh which has to be installed manually, TSL materials are able to be automatically applied (Espley et al. 2004). As such, higher roadway development rates can be achieved. In addition, as remote spraying of the TSL is possible, exposure of miners to unsupported roof can be avoided and thus roof fall injuries can be reduced.

(v) Penetration into the cracks and fissures by the TSL material contributes to sealing and reinforcement of fracture surfaces (Archibald 2001). Unlike steel mesh, TSL material can be sprayed into the cracks of rock mass, gluing the fractured rock and improving its strength.

(vi) TSL is normally applied as a thin layer of around 4 mm. It has significant benefit in terms of transporting over other forms of traditional surface support techniques (Archibald & Nicholls 2000). Compared with steel mesh and shortcrete, TSL application requires minimal material consumption (Moreau et al. 2001).

(vii) Some TSL products are highly light reflective, which provides better worker illumination, and some TSLs have the capability to restrict gas inflows and optimise flow capacities of ventilation networks, offering additional worker health benefits (Archibald 2001; Laurence 2004).

(viii) TSL is able to provide immediate support in advancing or reconditioning headings, which is important to miners’ safety (Moreau et al. 2001).
TSL generally provides these advantages: fast setting times, good tensile strength, excellent elongation properties and good bond strength (Pappas et al. 2003). Fast setting time is critical as it enables TSLs to resist rock deformation at an early stage, contributing to the overall stability of the strata.

As the TSL is in intimate contact with the rock surface, it performs better than mesh in bridging joints and fractures and restricting movement between adjacent blocks (Naismith & Steward 2001).

TSL is able to seal the rock from weathering and assist small blocks to interlock (Hepworth & Lobato 2002). It is also able to extend the influencing area of rockbolt and cable faceplates (Stacey 2001).

2.4.2.2 Disadvantages of thin spray-on liners

The disadvantages of the TSL are listed as follows:

(i) Where the rock is covered in dust, TSL may not work when it is impossible to create good adhesion between the liner and the dusty rock (Tannant 2004b).

(ii) Polymer-based TSL agents are typically electrical insulators which can accumulate static electricity (Lukey et al. 2008).

(iii) Direct exposure to some TSL materials may lead to allergic sensitisation. Hazard potential exists for exposure to either chemical or dust released during spraying (Archibald 2001).

(iv) Most TSL materials have a shelf life of only three to six months and exceeded shelf life results in strength reduction (Pappas et al. 2003).

(v) Most TSL materials specify storage temperature range and dry storage conditions, and it may be difficult to maintain a dry environment underground (Pappas et al. 2003).

2.4.3 Determining the mechanical properties of a thin spray-on liner

Espley-Boudreau (1999) defined four types of potential failure modes of thin spray-on liners; adhesive failure, tensile failure, direct shear failure and diagonal tensile failure. These failure modes are controlled by the mechanical properties of the TSL.
such as adhesion strength, tensile strength and shear strength. Prior to the application of the TSL material it is necessary to assess its mechanical properties.

2.4.3.1 Tensile properties investigation

Tensile strength is one of the fundamental mechanical properties of the TSL. The tensile strength of TSLs helps to promote block interlock and prevent block displacement. TSLs act primarily in tension once the failed rock induces the formation of basket (Stacey & Yu 2004). Tensile failure may occur if large rock displacement results in a great tensile stress greater than the tensile strength of the TSL material (Espley-Boudreau 1999). Tensile tests on TSLs have been carried out by many scholars (Tannant 1998; Archibald 2004; Kuijpers et al. 2004; Yilmaz 2010; Ahn 2011) to evaluate the tensile properties of various TSL materials.

Table 2.7 summarises the method used and properties measured by the above researchers. It is clear from the table that all of the tests were performed in accordance with the American Society for Testing and Materials test standard (ASTM D638-Standard Test Method for Tensile Properties of Plastics).

<table>
<thead>
<tr>
<th>Researchers</th>
<th>Test method</th>
<th>Properties measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mercer (1992)</td>
<td>ASTM D638</td>
<td>Tensile strength, elongation and Young’s modulus</td>
</tr>
<tr>
<td>Tannant (1998)</td>
<td>ASTM D638</td>
<td>Tensile strength</td>
</tr>
<tr>
<td>Kuijpers et al. (2004)</td>
<td>ASTM D638</td>
<td>Tensile strength, elongation and deformation stiffness</td>
</tr>
<tr>
<td>Archibald (2004)</td>
<td>ASTM D638</td>
<td>Tensile strength, elongation and modulus of deformation</td>
</tr>
<tr>
<td>Yilmaz (2010)</td>
<td>ASTM D638</td>
<td>Tensile strength</td>
</tr>
<tr>
<td>Ahn (2011)</td>
<td>ASTM D638</td>
<td>Tensile strength and elongation</td>
</tr>
</tbody>
</table>
Mercer (1992) stated that while the humidity had an adverse influence on the tensile strength of a TSL product, it didn’t affect the Young’s modulus and elongation of the TSL. Tannant’s study in 1998 (cited in Espley-Boudreau 1999) reported the tensile strength of a TSL material (Rockguard-I) was 14.3 MPa which was much higher than the tensile strengths of the TSL products studied by Kuijpers et al. (2004). In the study of Kuijpers et al. (2004), five TSL products of three different types (cement based, acrylic based and polyurethane based) were tested. The greatest tensile strength was achieved by the polyurethane based product, with a value of around 5.4 MPa. Archibald (2004) assessed the tensile and elongation properties of four types of TSL material (Mineguard™, RockGuard™, Rockweb and Masterseal®), the test results are listed in Table 2.8. Differences in the load-deflection behaviour of the four products were observed, which was believed to be attributed to the various thicknesses of the materials. The variability in thickness and material composition factors also produced different elongation capacities of the four types of TSL material. Note that the tensile strengths of Mineguard™ and Rockweb could not be determined as the test machine ran out of stroke due to significant elongation capacities. The stroke issue was also encountered in the study of Ahn (2011) where two of the four samples reached the maximum displacement of the test machine.

Table 2.8 Tensile properties of spray-on liner materials (Archibald 2004)

<table>
<thead>
<tr>
<th>Material</th>
<th>Tensile strength (MPa)</th>
<th>Elongation (%)</th>
<th>Modulus of deformation (MPa)</th>
<th>Range of liner thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mineguard™</td>
<td>15.58*</td>
<td>9.78</td>
<td>+200*</td>
<td>94.72</td>
</tr>
<tr>
<td>RockGuard™</td>
<td>11.36</td>
<td>8.78</td>
<td>86.69</td>
<td>61.28</td>
</tr>
<tr>
<td>Rockweb</td>
<td>13.05*</td>
<td>9.00</td>
<td>+200*</td>
<td>61.56</td>
</tr>
<tr>
<td>Masterseal®</td>
<td>2.50</td>
<td>-</td>
<td>65.91</td>
<td>-</td>
</tr>
</tbody>
</table>

* All specimens exhibited 50 mm limit of extension, with no break of test sample occurring
Yilmaz (2010) conducted a series of tensile tests on various TSL products over different curing periods. The tests were done following ASTM D638 with some modifications in the loading rate as most of the TSLs have limited deformation and would fail within 12 seconds if the recommended loading rate (5 mm/minute) was used. Figure 2.16 illustrates typical load-displacement behaviours of a plain and fibre reinforced TSL. The plain TSL failed in a brittle way after reaching its peak load while the fibre reinforced TSL experienced a plastic failure and exhibited the post failure behaviour. For underground coal mine roof support material, plastic failure is favoured as it has visual indications before final failure. As shown in Figure 2.17, it was found in this study that most of the TSL products tended to increase in tensile strength over the curing period.

It is important to note that although tensile properties of TSLs, such as tensile strength, elongation and Young’s modulus, have been evaluated frequently, there is little data on the Poisson’s ratio of TSLs which is a very important parameter in numerical modelling.

Figure 2.16 Typical load-displacement behaviours of plain and fibre reinforced TSLs (Yilmaz 2010)
2.4.3.2 Flexural properties investigation

Like tensile properties, flexural properties are also important properties of TSLs, however, the study on the flexural properties of the TSL is scarce. The flexural rigidity of TSLs is able to resist the deflection of the liner, which helps to prevent the initial movement of rock (Stacey & Yu 2004). In a proof of concept study, Lukey et al. (2008) investigated the flexural properties of reinforced polymers by conducting 3-point bending tests on rectangular beams. Load versus deformation curves of samples during the tests are shown in Figure 2.18. It is obvious that all of the samples failed in a plastic failure mode and exhibited post failure behaviour, which is desirable for roof control as presented in the above section. While some samples were strong but relatively brittle, others were less strong but more flexible. It is ideal to develop a TSL which has the strength of the former and the flexibility of the latter.
2.4.3.3 Adhesion properties investigation

2.4.3.3.1 Previous adhesion tests

Adhesion strength of the TSL is also termed tensile bond strength by some researchers. It is the capability of the TSL to adhere to the rock substrate when the rock-TSL interface is subjected to a normal tensile stress (Kuijpers et al. 2004). Archibald (2004) stated that ‘where adequate adhesion bonds exist, liners have the potential to transfer or carry load, created by gravity falls of loose rock in contact with liners, onto stable or un-fractured rock surfaces which also maintain liner contact’. Adhesion strength is one of the most important mechanical properties of the TSL. It has been studied by many researchers using laboratory and in situ experiments. Table 2.9 summaries the previous adhesion tests carried out. The test method used, size of testing area, substrate to which the TSL was adhered, loading rate and the thickness of the TSL are also presented.
Table 2.9 Previous adhesion test methods proposed and adhesion tests conducted

<table>
<thead>
<tr>
<th>Researchers</th>
<th>Method</th>
<th>Size of testing area</th>
<th>Substrate</th>
<th>Loading rate</th>
<th>TSL thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mercer (1992)</td>
<td>Embedded dolly pull</td>
<td>125 mm, 250 mm diameter</td>
<td>Concrete, granite, sandstone &amp; limestone</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Tannant et al. (1999)</td>
<td>Embedded dolly pull</td>
<td>58.7 mm diameter</td>
<td>Concrete</td>
<td>About 1 mm/min</td>
<td>N/A</td>
</tr>
<tr>
<td>Lewis (2001)</td>
<td>Glued dolly pull</td>
<td>N/A</td>
<td>Granite</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Spearing and Gelson (2002)</td>
<td>Embedded dolly pull</td>
<td>N/A</td>
<td>Concrete</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Kuijpers et al. (2004)</td>
<td>Glued dolly pull</td>
<td>39 mm</td>
<td>Norite &amp; rock</td>
<td>&lt; 1 MPa/5s</td>
<td>1-7 mm</td>
</tr>
<tr>
<td>Spearing et al. (2004)</td>
<td>Core to core pull</td>
<td>N/A</td>
<td>Rock</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Ozturk &amp; Tannant (2004, 2010, 2011); Ozturk (2005);</td>
<td>Glued dolly pull</td>
<td>33 mm diameter</td>
<td>Concrete, limestone, granite, cinder &amp; sandstone</td>
<td>0.1 mm/min, 0.5 mm/min, 1 mm/min, 2 mm/min</td>
<td>1-15 mm</td>
</tr>
<tr>
<td>Archibald (2004)</td>
<td>Embedded dolly pull</td>
<td>100 mm diameter</td>
<td>Paving stone</td>
<td>1 mm/min</td>
<td>2-4 mm</td>
</tr>
<tr>
<td>Espley et al. (2004)</td>
<td>Embedded dolly pull</td>
<td>N/A</td>
<td>Rock &amp; shotcrete</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Nemcik et al. (2009)</td>
<td>Centre push</td>
<td>100 mm × 100 mm square</td>
<td>Sandstone and coal</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Ahn (2011)</td>
<td>Embedded dolly pull</td>
<td>300 mm diameter</td>
<td>Concrete &amp; Sika grout</td>
<td>0.5 mm/min</td>
<td>4 mm</td>
</tr>
<tr>
<td>Yilmaz (2011)</td>
<td>Glued dolly pull</td>
<td>35 mm diameter</td>
<td>Norite</td>
<td>2.5 N/s to 100 N, then 5 N/s</td>
<td>5 mm</td>
</tr>
<tr>
<td>Li et al. (2015)</td>
<td>Glued dolly pull</td>
<td>28.2 mm diameter</td>
<td>Coal</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>


Unlike tensile test, there is no standard test method particularly suitable for measuring the adhesion strength of TSLs. Researchers have proposed many methods to evaluate the adhesion of a TSL, which include core to core pull test, centre push test, embedded
dolly pull test and glued dolly pull test as presented in Table 2.9. Figure 2.19 illustrates the schematics of the adhesion test methods.

The core to core pull test (Spearing et al. 2004), as shown in Figure 2.19a, involves two pieces of rock core which are bonded together with TSL. During the test, the top and bottom cores are subjected to a uniaxial pull force until failure occurs at the rock-TSL interface. It is worthwhile to note that there are no data on this test available. The core to core pull test method is simple, practical and cost effective, however, this test method has an obvious disadvantage in that the inner TSL material may cure slower than the external portion. There is the possibility that the centre portion of the TSL may not even cure at the time of the testing (Potvin, Stacey & Hadjigeorgiou 2004).
The centre push test is schematically shown in Figure 2.19b. In this test, the TSL was initially cast onto the flat surface of the substrate which was drilled through in the central beforehand, and then it was allowed to cure. During the test, the liner was pushed downwards in the middle through the pre-drilled hole in the substrate by the use of a plunger till the liner material detaches from the substrate. A number of tests were conducted by Nemcik et al. (2009) to evaluate the adhesion of a TSL to sandstone and coal surfaces using the centre push test method. The test results are shown in Table 2.10. It was found that all the coal surface samples experienced tensile failure in coal, indicating the adhesion of the TSL to coal was greater than the tensile strength of coal. As sandstone was stronger than coal in tensile strength, adhesion failure occurred at the TSL-sandstone interface with the maximum tensile load of 103 kPa.

Table 2.10 Adhesion of a polymer skin to rock surface (Nemcik et al. 2009)

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Surface preparation</th>
<th>Max tensile load (kPa)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal</td>
<td>Dry &amp; clean</td>
<td>69</td>
<td>Tensile failure in coal</td>
</tr>
<tr>
<td>Coal</td>
<td>Wet pH 1</td>
<td>48</td>
<td>Tensile failure in coal</td>
</tr>
<tr>
<td>Coal</td>
<td>Wet pH 7</td>
<td>40</td>
<td>Tensile failure in coal</td>
</tr>
<tr>
<td>Coal</td>
<td>Wet pH 13</td>
<td>40</td>
<td>Tensile failure in coal</td>
</tr>
<tr>
<td>Sandstone</td>
<td>Dry &amp; clean</td>
<td>103</td>
<td>Rock-polymer bond failure</td>
</tr>
</tbody>
</table>

The glued dolly pull test (Figure 2.19c) consists of the substrate, the TSL material, the epoxy and the dolly. In this test, the TSL material was first applied onto the surface of the substrate. When the TSL cured, the dolly was glued onto the top surface of the TSL with epoxy. After that, a kerf was cut around the perimeter of the dolly to isolate the TSL directly beneath the dolly from the other portion. As this test method is simple, easy, practical and cost effective (Potvin, Stacey & Hadjigeorgiou 2004), it has been applied by many researchers to study the adhesion strength of TSLs, as shown in Table 2.9. However, this test method also has some shortcomings. For instance, eccentric loading or bending may occur as a result of incorrect setting up of the test equipment,
the adhesion strength can be calculated only if the failure takes place at the rock-substrate interface, and it is possible that the TSL and/or the TSL-substrate interface are damaged by the overcoring process (Potvin, Stacey & Hadjigeorgiou 2004). It is important to notice that Yilmaz (2011) eliminated the overcoring process by making the substrate top surface the same size as the bottom surface of the dolly. With this method, samples failed in various failure modes during the adhesion test. By visually estimating the percentage of substrate left on the liner, Ozturk and Tannant (2004) classified the failure modes into three different types, which are shown in Table 2.11, while Yilmaz (2011) observed four different failure types which are presented in Table 2.12.

Table 2.11 Classification of failure mode (after Ozturk & Tannant 2004)

<table>
<thead>
<tr>
<th>Substrate left on liner (%)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-33</td>
<td>Adhesive at interface</td>
</tr>
<tr>
<td>34-66</td>
<td>Combination</td>
</tr>
<tr>
<td>67-100</td>
<td>Cohesive in the substrate</td>
</tr>
</tbody>
</table>

Table 2.12 Classification of failure mode (after Yilmaz 2011)

<table>
<thead>
<tr>
<th>Failure modes</th>
<th>Descriptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I failure</td>
<td>Substrate is pulled out by the TSL and accounts for more than 50% of the pull area</td>
</tr>
<tr>
<td>Type II failure</td>
<td>De-attachment occurs at the substrate-TSL interface with little TSL material adhered to the substrate</td>
</tr>
<tr>
<td>Type III failure</td>
<td>Failure occurs within the TSL</td>
</tr>
<tr>
<td>Type IV failure</td>
<td>De-bonding occurs at the TSL-epoxy interface or epoxy-dolly interface</td>
</tr>
</tbody>
</table>

It is clear from the two tables that the adhesive failure at interface in Ozturk and Tannant’s (2004) classification is similar with the type II failure in Yilmaz’s (2011)
classification. The difference lies in that the former defined the exact percentage while the latter did not. Likewise, the cohesive failure in the substrate in Ozturk and Tannant’s (2004) classification is almost identical to the type I failure in Yilmaz’s (2011) classification, except for the determination of the amount of substrate left on the TSL.

The embedded dolly pull test (Figure 2.19d) was similar to the glued dolly pull test except that the dolly is embedded in the liner material. With respect to the sample preparation, an initial layer of liner material was firstly applied onto the substrate and immediately followed by placing the perforated dolly onto the liner coating. As such, the liner material was able to seep through the perforation holes. And then, a second layer of liner material was applied onto the dolly to fully embed the dolly. The first and second liner layer should be at least 1 and 2 mm respectively (Archibald 2004). This test method was also easy and cost effective, but it was difficult to calculate an accurate adhesion strength as a result of the perforation holes (Potvin, Stacey & Hadjigeorgiou 2004). In addition, the adhesion of the TSL is affected by its thickness (Ozturk & Tannant 2010), however, it is difficult to determine the effective thickness of the TSL in this method. Four types of failure mode were observed by Archibald (2001) (cited in Kuijpers et al. 2004) during the test, which included: (1) partial failure of bottom layer of the liner with the material attached to the substrate after failure of the top layer, which resulted from poor anchorage due to a thin top liner; (2) full debonding at the substrate-TSL interface; (3) shear failure of the liner at the interface between the perforated plate and the bottom liner layer due to the tensile strength of the material being weaker than the adhesive strength; (4) failure of the liner material with substrate fragments attached, which is because the adhesive strength is greater than the tensile strength of the substrate.

2.4.3.3.2 Factors influencing the adhesion of thin spray-on liners

The factors that affect the adhesion of TSLs to the substrate have been widely studied and are presented as follows:

(i) substrate type;

(ii) surface contaminants;
(iii) substrate surface moisture;
(iv) cure time;
(v) liner thickness;
(vi) loading rate.

**Substrate type**

TSLs are designed to adhere to the rock surface in underground coal mines. As rock substrate varies, it is important to investigate the sensitivity of the TSL adhesion to different rock types. Ozturk and Tannant (2011) studied the adhesion strength of a cement-based TSL material (Tekflex) onto various types of substrate using glued dolly pull tests in the laboratory. Table 2.13 presents the properties of various substrates and Table 2.14 shows the laboratory test results.

Table 2.13 Substrate tensile strength, surface roughness and average grain size (Ozturk & Tannant 2011)

<table>
<thead>
<tr>
<th>Sample</th>
<th>No. of test</th>
<th>Tensile strength (MPa)*</th>
<th>Average grain size (mm)</th>
<th>Surface condition</th>
<th>Roughness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete cinder block</td>
<td>9</td>
<td>1.6 (0.4)</td>
<td>2</td>
<td>Natural</td>
<td>1</td>
</tr>
<tr>
<td>Paving stone</td>
<td>15</td>
<td>3.7 (0.62)</td>
<td>3</td>
<td>Sawn or natural</td>
<td>1</td>
</tr>
<tr>
<td>Berea Sandstone</td>
<td>9</td>
<td>1.5 (0.6)</td>
<td>0.2</td>
<td>Split</td>
<td>20</td>
</tr>
<tr>
<td>Sandstone</td>
<td>4</td>
<td>11.4 (3.9)</td>
<td>0.6</td>
<td>Split</td>
<td>20</td>
</tr>
<tr>
<td>Granite</td>
<td>9</td>
<td>10.4 (2.8)</td>
<td>1.8</td>
<td>Split</td>
<td>20</td>
</tr>
<tr>
<td>Limestone</td>
<td>5</td>
<td>2.7 (0.6)</td>
<td>0.2</td>
<td>Split</td>
<td>20</td>
</tr>
</tbody>
</table>

* Mean (standard deviation).

It was demonstrated that while rock tensile strength and grain size tended to have a positive influence on the adhesive strength, rock surface roughness seemed to have negligible effect. Awaja et al. (2009) stated that there has been no coincident agreement regarding the influence of interface surface roughness on the polymer
adhesion, while some researchers (Morris et al. 1998) argue that a rough surface results in mechanical interlocking and produces higher adhesion strength and others (Basin 1984; Vasconcelos et al. 2004) suggest that a rough surface provides more area for molecular bonding.

Table 2.14 Adhesive strengths for different substrates (Ozturk & Tannant 2011)

<table>
<thead>
<tr>
<th>Sample</th>
<th>No. of test</th>
<th>Adhesion (MPa)*</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Split granite</td>
<td>4</td>
<td>1.3 (0.3)</td>
<td>Interface</td>
</tr>
<tr>
<td>Saw cut granite</td>
<td>3</td>
<td>1.4 (0.1)</td>
<td>Interface</td>
</tr>
<tr>
<td>Split limestone</td>
<td>3</td>
<td>1.4 (0.3)</td>
<td>Interface &amp; substrate</td>
</tr>
<tr>
<td>Split Berea sandstone</td>
<td>4</td>
<td>0.7 (0.1)</td>
<td>Interface</td>
</tr>
<tr>
<td>Paving stone</td>
<td>4</td>
<td>1.8 (0.2)</td>
<td>Interface</td>
</tr>
<tr>
<td>Cinder block</td>
<td>2</td>
<td>0.8 (0.0)</td>
<td>Interface</td>
</tr>
</tbody>
</table>

* Mean (standard deviation).

Surface contaminants

In underground mines, the excavation surface may be contaminated with dust or oil before the application of TSL. Thus, it is necessary to evaluate the effect of contaminants on the adhesive strength of TSL. This influence was studied by Ozturk and Tannant (2011) and the results are shown in Table 2.15. It was found that the adhesion of a cement-based TSL (Tekflex) onto the rock substrate was detrimentally affected by the dust. The effect was proportional to the amount of the contamination. Molecular bonding is one of the adhesion mechanisms, this mechanism requires intimate contact between the two surfaces, defects, cracks and air bubbles have a negative effect on the adhesion (Awaja et al. 2009). The presence of contaminant material resulted in less contact area between the TSL and substrate thus reducing the adhesion strength. For porous substrates, such as cinder blocks, the dust decreased the adhesion much more than the oil did. This was probably because the oil was able to penetrate into the pores while the dust stayed on the surface of the block.
Table 2.15 Effect of surface contaminants on adhesive strength – mean (standard deviation) (after Ozturk & Tannant 2011)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Amount of contaminant</th>
<th>No. of test</th>
<th>Adhesion (MPa)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dusty paving stone</td>
<td>0.005 g/cm²</td>
<td>4</td>
<td>0.2(0.1)</td>
<td>Interface</td>
</tr>
<tr>
<td>Dusty paving stone</td>
<td>0.0025 g/cm²</td>
<td>4</td>
<td>0.9(0.1)</td>
<td>Interface</td>
</tr>
<tr>
<td>Dusty paving stone</td>
<td>0.001 g/cm²</td>
<td>4</td>
<td>1.5(0.1)</td>
<td>Interface</td>
</tr>
<tr>
<td>Oily paving stone</td>
<td>0.03-0.1cm³/ cm²</td>
<td>8</td>
<td>0.0</td>
<td>Interface</td>
</tr>
<tr>
<td>Oily paving stone</td>
<td>0.008 cm³/ cm²</td>
<td>4</td>
<td>0.0-0.7</td>
<td>Interface</td>
</tr>
<tr>
<td>Dusty cinder block</td>
<td>0.005 g/cm²</td>
<td>4</td>
<td>0.5(0.1)</td>
<td>Interface</td>
</tr>
<tr>
<td>Oily cinder block</td>
<td>0.1 cm³/ cm²</td>
<td>3</td>
<td>1.0(0.3)</td>
<td>Interface</td>
</tr>
</tbody>
</table>

Substrate surface moisture

Apart from dust and oil, moisture is also one of the most common environmental conditions that TSLs may be exposed to in underground coal mines. It is reasonable to believe that the adhesion strength of a TSL is adversely affected by the substrate surface moisture. This was confirmed by Espley et al. (2004) who performed embedded dolly pull tests on a cement-based TSL material with various levels of substrate moisture. The relative percent surface moisture was measured with a Tramex™ surface moisture metre. The test results are listed in Table 2.16. It was shown that the adhesion strength increased as the surface moisture level decreased. According to Ferguson (2004), moisture can have an influence on the interfacial adhesion not only in a direct way but also in an indirect way. To be specific, the moisture can be physically present at the interface preventing intimate contact between the adhesive and the substrate and thus weakening the molecular bonding. It is also
able to alter the mechanical properties of the adhesive and substrate as a result of moisture uptake. It is important to note that moisture was found to increase the adhesion of Tekflex which is a type of cement based TSL product (Ozturk 2005).

Table 2.16 Adhesive strength versus surface moisture (Espley et al. 2004)

<table>
<thead>
<tr>
<th>Cure time (hr)</th>
<th>Surface moisture</th>
<th>Peak stress (MPa)</th>
<th>Substrate material</th>
<th>Comments: failure mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>48%</td>
<td>0.66</td>
<td>Rock</td>
<td>Pure adhesion failure</td>
</tr>
<tr>
<td>2</td>
<td>50%</td>
<td>0.49</td>
<td>Rock</td>
<td>Tensile failure</td>
</tr>
<tr>
<td>2</td>
<td>60%</td>
<td>0.24</td>
<td>Rock</td>
<td>Pure adhesion failure</td>
</tr>
<tr>
<td>8</td>
<td>45%</td>
<td>0.58</td>
<td>Rock</td>
<td>Pure adhesion failure</td>
</tr>
<tr>
<td>8</td>
<td>45%</td>
<td>0.56</td>
<td>Rock</td>
<td>Pure adhesion failure</td>
</tr>
<tr>
<td>8</td>
<td>80%</td>
<td>0.25</td>
<td>Shotcrete</td>
<td>Pure adhesion failure</td>
</tr>
<tr>
<td>8</td>
<td>80%</td>
<td>0.30</td>
<td>Shotcrete</td>
<td>Pure adhesion failure</td>
</tr>
<tr>
<td>24</td>
<td>45%</td>
<td>0.67</td>
<td>Rock</td>
<td>5% tensile, 70% adhesion and 25% broken rock</td>
</tr>
<tr>
<td>24</td>
<td>50%</td>
<td>0.51</td>
<td>Rock</td>
<td>15% tensile, 60% adhesion and 25% broken rock</td>
</tr>
</tbody>
</table>

**Cure time**

With respect to the influence of cure time on the adhesive strength of TSLs, inconsistent results have been reported. Espley et al. (2004) studied the effect of the cure time on adhesion by conducting field tests. The results are presented in Table 2.16. It was shown that the cure time did not affect the adhesion strength of the TSL but it had a significant effect on the failure mechanism. When the cure time was 8 hours or less almost all of the samples experienced pure adhesion failure, however, when the samples were cured for 24 hours the failure mechanism was mixed, with the main failure occurring at the interface. Ferguson (2004) suggested that this was
probably due to the longer curing time assisting moisture uptake which in turn changed the properties of the adhesive and substrate. In the study of Ozturk (2005) it was indicated (Table 2.17) that longer curing time produced greater adhesion strength. One of the possible reasons for the contrasting results from Espley et al. (2004) and Ozturk (2005) could be different TSL materials were used.

Table 2.17 Measured adhesion of Tekflex (mean & standard deviation in MPa) after 1 week and 1 month of curing time (Ozturk 2005)

<table>
<thead>
<tr>
<th>Sample</th>
<th>No. of tests</th>
<th>1 week</th>
<th>No. of tests</th>
<th>1 month</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saw cut granite</td>
<td>4</td>
<td>1.3 (0.1)</td>
<td>3</td>
<td>2.5 (0.3)</td>
</tr>
<tr>
<td>Clean paving stone</td>
<td>5</td>
<td>1.9 (0.2)</td>
<td>4</td>
<td>3.4 (0.2)</td>
</tr>
<tr>
<td>Damp paving stone</td>
<td>2</td>
<td>2.1 (0.2)</td>
<td>3</td>
<td>2.5 (0.6)</td>
</tr>
</tbody>
</table>

**Liner thickness**

While most of the researchers did not observe the effect of TSL thickness on its adhesive strength, Ozturk and Tannant (2010) found that the adhesion strength had an inverse square root relationship with the liner thickness (Figure 2.20). The relationship can be expressed by the following equation:

\[
\sigma_a^2 = \frac{2E\lambda}{t(1-\nu^2)}
\]  

(2.3)

Where:

\( \sigma_a \) = adhesion strength (MPa)

E = Young’s modulus (MPa)

\( \lambda \) = work of adhesion (N/mm)

t = liner thickness (mm)

\( \nu \) = Poisson’s ratio

The work of adhesion is defined as the energy required to separate a unit area of the contacting surfaces (Ozturk & Tannant 2010). This finding is supported by the study
of Kuijpers et al. (2004) in which it was discovered that adhesion strengths of two types of TSL product increased as the thickness decreased. Awaja et al. (2009), in reporting the studies of Basin (1984), Chen et al. (2007) and Voyutskii (1963), stated that excessive chemical bonds can result in stress concentration at the interface decreasing the interface strength, i.e. the interface bonding strength decreases as the thickness of the adhesive layer is increased because it is harder for the stress to dissipate through the interface.

![Figure 2.20 Adhesive strength versus TSL thickness for two substrates (Ozturk & Tannant 2010)](image)

**Loading rate**

The effect of loading rate on the adhesion strength of a TSL was investigated in the study of Ozturk (2005) in which three different loading rates (0.1, 0.5 and 2 mm/min) were evaluated in the laboratory. The test results are shown in Figure 2.21. It can be seen that the influence of the applied loading rates on the TSL adhesion strength can be neglected.
The creep behaviour of a TSL material (Tekflex) was also investigated in this study. The samples were classified into two groups with one group cured for 7 days and the other group cured for 45 to 62 days. The two groups of sample had an applied stress from 0.66 to 1.1 MPa and 0.43 to 1.6 MPa respectively. The test results (as shown in Figure 2.22) indicated that, for both of the two groups of sample, the larger the applied stress the shorter the TSL could adhere.

2.4.3.4 Shear and shear-bond properties investigation

In underground surface support, a TSL may be subjected to shear force when loose rock block displace along a fracture or joint (Potvin, Stacey & Hadjigeorgiou 2004).
Penetration of TSL material into the fracture or joints helps to promote block interlock (Stacy & Yu 2004). The shear bond of the TSL plays a role in this case (Yilmaz 2011). Thus, it is necessary to determine the shear and shear bond strength of the TSL. Like adhesion strength, there has been no standard test method for measuring the shear strength or the shear bond strength. Many test methods have been developed and applied by researchers (Saydam et al. 2003; Yilmaz 2007; Yilmaz 2011; Qiao et al. 2014), and they are discussed in this section.

2.4.3.4.1 Shear strength test methods

The shear strength of TSLs has been evaluated by Yilmaz (2011) and Qiao et al. (2014), schematics of the test set-up proposed by them are shown in Figure 2.23 and Figure 2.24 respectively. There were mainly four apparatus in the test method of Yilmaz (2011). They were a steel support ring, a steel TSL holding ring, a clamping fixture which consists of two steel plates with superimposing holes with a diameter less that the holding ring and a steel punch. During the test, the TSL ring was sandwiched between the two steel plates and clamped tightly, then they were placed onto the support ring, the load was applied to the TSL with the punch through the hole in the top plate.

Figure 2.23 Schematic of shear strength test set-up (Yilmaz 2011)
Compared with Yilmaz’s (2011) method, Qiao et al.’s (2014) method involved less apparatus. The test apparatus consist of a clamping fixture which was composed of two steel plates with a hole in the middle and a steel punch. During the test the TSL sample was placed between the two steel plates and then clamped tightly, the load was applied with the steel punch. This test method was relatively easy as it was possible to conduct several tests on one TSL plate, decreasing the sample preparation time. Neither of the two studies (Yilmaz 2011; Qiao et al. 2014) investigated the influence of TSL thickness on its shear strength.

![Shear strength test set-up (Qiao et al. 2014)](image)

Both of the test methods have been successfully applied to study the shear strength of TSLs. Yilmaz (2011) found that the TSL material exhibited strength improvement over a 28 day curing period. It was observed in the study of Qiao et al. (2014) that fibre reinforcement contributed to shear strength increase, the more layers of fibre sheet that was embedded, the greater the shear strength.

### 2.4.3.4.2 Shear bond strength test methods

Two types of testing approaches were developed to study the shear bond strength of a TSL, a double sided shear (DSS) test (Saydam et al. 2003) and double ring punch (DRP) test (Yilmaz 2007). The DSS test system (Figure 2.25) consisted of three granite blocks with the adjacent surfaces glued by the TSL. During the test, the two side granite blocks were placed onto the base and clamped tightly. A gap between the base and the bottom surface of the central block was left. The load was applied on the
central block till the failure of the sample. This test represents the in situ case in which the TSL penetrates into the cracks, however, premature failure may occur due to bending if the side blocks are not clamped tightly (Potvin, Stacey & Hadjigeorgiou 2004).

Figure 2.25 Schematic of double sided shear test (Saydam et al. 2003)

The DRP test method (Figure 2.26) involves two 20 mm thick steel rings, a rock core and the TSL. Before the test, the rock core was placed at the centre of the steel ring and the gap between them was filled with the TSL. After the TSL cured for a predetermined period the sample was placed on the steel support ring which could support the TSL and the other steel ring but not the rock core. The load was applied by pushing the rock core downwards till failure occurs. As the rock core was circumferentially secured, this test method avoided the issue of specimen bending or rotation induced by the application of shear loading. This study found that there was an increase in the shear-bond strength of the TSL over a curing period of 28 days.

Figure 2.26 Schematics of shear bond strength test (Yilmaz 2007)
2.4.3.5 Load bearing capacity investigation

A thin spray-on liner, as a potential medium to replace steel mesh for underground roof support, must be strong enough to resist the gravity-induced load of fractured rock. Thus, it is important to evaluate the load bearing capacity of TSLs. Nemcik et al. (2011a) suggested that large scale laboratory tests were necessary to attempt to predict the performance of a TSL before in situ trials. Many researchers have conducted large scale laboratory tests to evaluate the load bearing capacities of TSLs. Table 2.18 lists the previous tests conducted and the detail of the tests are presented in this section.

<table>
<thead>
<tr>
<th>Researchers</th>
<th>Liner size tested</th>
<th>Liner thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tannant (1997)*</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Connor et al. (2003)</td>
<td>N/A</td>
<td>2-3 mm</td>
</tr>
<tr>
<td>Swan &amp; Henderson (2004)</td>
<td>1100 mm square</td>
<td>Nominal 6 mm</td>
</tr>
<tr>
<td>Kuijpers (2004)</td>
<td>600 mm square</td>
<td>2-6 mm</td>
</tr>
<tr>
<td></td>
<td>1750 mm square</td>
<td></td>
</tr>
<tr>
<td>Finn (2004)</td>
<td>2625 mm square</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>3500 mm square</td>
<td></td>
</tr>
<tr>
<td>Nemcik et al. (2009)</td>
<td>800 mm × 1000 mm</td>
<td>5 mm</td>
</tr>
<tr>
<td>Nemcik et al. (2011a)</td>
<td>800 mm × 600 mm</td>
<td>5 mm</td>
</tr>
</tbody>
</table>

Note: * - cited in Spearing et al. (2004)

2.4.3.5.1 Tannant (1997)

Figure 2.27 illustrates the schematic of the plate pull test conducted by Tannant (1997). In this test, interlocking hexagonal concrete paving blocks were sprayed with the TSL and then a testing frame was lifted onto the sample. A pull load was applied at the centre of the bottom face of the concrete blocks till failure of the TSL occurred. The test method involved the interaction between the substrate and the TSL, however, it
was difficult to maintain uniform liner thickness and the test result was poorly repeatable (Potvin, Stacey & Hadjigeorgiou 2004).

Figure 2.27 Schematic of the plate pull test conducted by Tannant (1997)

2.4.3.5.2 Connor et al. (2003)

Figure 2.28 describes the ‘bagging load test’ conducted by Connor et al. (2003). This test included five major elements, which were the TSL sheet, a layer of thin slabby granitic material, a layer of coarse gravel, a loading platen and the steel loading frame. Two types of test arrangement were investigated. There was no adhesion between the TSL material and the granitic material in the first type, the second type involved adhesion at the interface of the two materials. The test results indicated that adhesion had a significant positive effect on the load bearing capacity of the TSL. The information about the TSL size and the peak loads achieved in these tests were not provided in the paper.
2.4.3.5.3 Swan & Henderson (2004)

Swan & Henderson (2004) evaluated the load bearing capacity of a TSL product (Tekflex®) using the test method schematically described in Figure 2.29. An open-ended steel frame with dimensions of $1.1 \times 1.1 \times 0.3$ m was employed in this test. Unwashed 100 mm loose rock debris were poured into the steel frame and roughly levelled. Nominal 6 mm thick TSL was then sprayed onto the surface of the debris and cured for a predetermined period. During the test, the steel frame was inverted and the debris surface was compressed by a loading platen till failure of the TSL. The Tekflex® was observed to have a bearing capacity of 24 kN. As the liner was sprayed onto the substrate, it may be difficult to achieve consistent liner thickness for this test method.
2.4.3.5.4 Kuijpers (2004)

A series of large scale laboratory tests were conducted by Kuijpers (2004) to study the reinforcing effect of a TSL. Figure 2.30 describes the test method utilized. Concrete panels of 600 mm square and 100 mm thick were cast. Tests were conducted on non-coated panels and TSL (Evermine) coated panels with varying liner thickness. During the test, the load was applied at the centre of its top surface. The results indicated that while the peak load of the non-coated panel was around 18 kN that of the 4.5 mm thick TSL confined panel reached about 22 kN. When the concrete panel was coated with a thicker liner its energy absorption capacity improved.

Figure 2.30 Schematic of the test conducted by Kuijpers (2004)

2.4.3.5.5 Finn (2004)

Finn’s (2004) test method is shown in Figure 2.31. In this test, a steel plate was firstly placed onto a concrete floor and then sprayed over with the liner material. The load was applied by pulling the steel plate upwards. Three different sizes of the plate were used; 500 mm square, 750 mm square and 1000 mm square. For each plate size, the sprayed liner was extended by 1.25 times the plate side length from the edges. The tests were conducted on three types of TSL material which included SPI polyuria, Kohlbergerer polyurethane and MBT Masterseal®. Generally, Kohlbergerer polyurethane was the strongest and MBT Masterseal® was the weakest. The peak loads of Kohlbergerer polyurethane subject to 500 mm square, 750 mm square and 1000mm square plate were around 35 kN, 38 kN and 28 kN respectively. The
corresponding strengths for SPI polyuria were 17 kN, 23 kN and 33 kN respectively. The strength of Kohlbergerer polyurethane with 1000 mm square plate was lower than that of 750 mm square plate, which was because the TSL with 1000 mm square plate failed due to delamination of the liner layers. While the Kohlbergerer polyurethane samples failed in a brittle way, the SPI polyuria liners had significant post failure behaviour with a residual strength of around 13-20 kN over a displacement of about 100 mm. It was concluded that SPI polyuria had the potential to replace steel mesh for roof support provided it did not produce isocyanates during the spraying process.

![Schematic of the test conducted by Finn (2004)](image)

Figure 2.31 Schematic of the test conducted by Finn (2004)

2.4.3.5.6 Nemcik et al. (2009)

Nemcik et al (2009) compared the load bearing capacities of a TSL with welded steel mesh. The test is schematically shown in Figure 2.32. During the test, the TSL and steel mesh were clamped in an open-ended timber frame and gradually loaded with terracotta pavers. The test terminated when the load achieved 10 kN while neither of the TSL nor the steel mesh failed. As such, it was not possible to compare the peak loads the two support mediums were able to bear, however, the test results indicated that the TSL had a smaller deflection (20 mm) than the steel mesh (35 mm). Furthermore, 90% of the TSL deflection recovered after the load was removed while the deflection recovery percentage of steel mesh was only 60%.
2.4.3.5.7 Nemcik et al. (2011a)

As the 10 kN load was not enough to fail either the TSL or the steel mesh sheet, Nemcik et al. (2011a) conducted four more tests to determine the ultimate load bearing capacity of the TSL using a 500 t Avery compressive testing machine. Instead of the timber frame, a steel frame was used during the tests. The size of the test sheet was 800 mm × 600 mm due to the limitation of the test machine size, and its thickness was 5 mm.

Figure 2.33 illustrates the schematics of the large scale laboratory tests. In the first test, an air bag was placed on the top of the 5 mm thick TSL sheet to achieve even load distribution. The test terminated at a certain point without TSL failure occurring as the machine ran out of its stroke, which was due to the substantial decrease of volume of the airbag as a result of the compressive load.

As the TSL did not fail in the first test, a smaller loading area was applied to determine its load bearing capacity. A 150 mm diameter steel spherical seat was employed to load the TSL. In order to eliminate the effect of stress concentrations a rubber mat was placed between the steel seat and the TSL. Failure occurred in this test at 45 kN which
was smaller than the maximum load achieved in test 1. This was expected as the load was more concentrated in test 2, which was confirmed with the fact that the failure location was directly beneath the spherical seat.

In order to determine the load capacity of the TSL under distributed loading conditions, a number of terracotta pavers were bonded to the TSL sheet and an airbag was placed onto the pavers in test 3. The test stopped without failure when the load was at 110 kN, as the authors were concerned that the airbag might explode due to excessive air pressure. The much higher peak load achieved in this test indicated that the pavers had a remarkable effect on load distribution.

Test 4 was conducted with the same conditions as in test 2 except that three layers of pavers were used to evenly distribute the load. At first, the pavers were bonded to the polymer, unfortunately adhesion was lost during the loading. Test 4 had similar results to test 2.

Figure 2.33 Schematic of the test conducted by Nemcik et al. (20011a)
2.4.4 Modelling strata & support interaction

Numerical modelling is a very effective research method in dealing with geotechnical engineering problems. It has many advantages such as: it can address complicated issues, it is able to measure the parameters which cannot be monitored in physical experiments and it is also cost effective. The behaviour of TSLs in strata control has been studied by many researchers using numerical simulations. A brief description of these studies is presented in the following:

**Connor et al. (2003)**

In this study, the numerical modelling program FLAC (Fast Lagrangian Analysis of Continua) was applied to study the behaviour of a TSL for rock support. The results from the numerical modelling were compared to that from the ‘bagging load test’ as discussed previously (Figure 2.28). It was found that while adhesion existed between the TSL and the overlying granitic material the total displacement obtained from the numerical modelling was much greater than that from the laboratory test. The authors believed that the discrepancy could have resulted from three possible sources. Firstly, the FLAC modelling was two dimensional instead of three dimensional as the physical test. Secondly, the effect of the adhesion could not be fully reflected in the numerical model. Lastly, it was difficult to account for the interlock of the slabby blocks in the FLAC model.

**Wang and Tannant (2004)**

In this study, the authors investigated the influence of a thin tunnel liner for ground control. The program used was PFC$^{2D}$ (Particle Flow Code in two dimensions). Two models were created, one had liner material adhered to the excavation surface of the underground tunnel while the other one did not. Both the rock and the liner material were modelled with particles which were rigid but deform locally at contact points. This was fulfilled by the use of a soft contact, in which finite normal and shear stiffness were used. It was observed from the numerical modelling that the low stiffness liner was not able to prevent cracks generating and developing in the rock around the tunnel. However, the liner was capable of keeping the fractured rock in place and controlling or decreasing deformation of the rock around the tunnel, which ultimately benefited
rock stability. The authors suggested that a similar numerical simulation would also work well with a stiffer and thicker liner such as shotcrete.

**Dirige and Archibald (2009)**

In this study, numerical modelling was conducted to gain a better understanding of the support performance of TSL under highly stressed mining environment conditions using FLAC\textsuperscript{3D} (Fast Lagrangian Analysis of Continua in three dimensions). The numerical model created was a half circular tunnel, only one-half of the tunnel was modelled due to symmetry. The rock was modelled as a Mohr-Coulomb strain softening material and the liner was modelled with shell structural elements. The modelling results indicated that a TSL (3.5 mm thick) may have equal or even better support capacity than shotcrete. Specifically, when there was no liner reinforcement, the tunnel crown and sidewall underwent 66 and 65 mm displacement respectively at 3000 time steps, and they tended to keep displacing as indicated in the displacement plots, however, when the tunnel was coated with TSL, the corresponding displacements were reduced to 46 and 58 mm respectively. Moreover, the displacement plots stalled. The crown and sidewall displacements were around 52 and 60 mm respectively at 3000 time steps when the tunnel was reinforced with 100 mm thick shotcrete.

It can be deduced from the above references that, as a 2 dimensional modelling program FLAC may not be able to deal with some 3 dimensional problems relating to TSL support behaviour. PFC\textsuperscript{2D} may be more suitable for analysing thicker and stiffer liner, while FLAC\textsuperscript{3D} appears to be the most suitable code to evaluate TSL performance in underground mining.

2.4.5 Field trials of the thin spray-on liner in coal mines

While small scale and large scale laboratory tests are useful to predict the possible behaviour of TSL material in underground coal mines, field tests are necessary to assess the in situ performance of the TSL. The field trials of TSLs have been studied by many researchers and are presented in Table 2.19. It can be seen from the table
that the TSLs were generally able to provide areal support, however, there has been no publication on the performance of TSLs in supporting large underground excavation surfaces.

<table>
<thead>
<tr>
<th>Researchers</th>
<th>TSL thickness / Test area / Test duration / Test area condition</th>
<th>Test results (TSL used)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kuijpers (2004)</td>
<td>3 mm / 16 m section of pillar sidewall / 1 month / Serious scaling problems</td>
<td>TSL provided excellent areal coverage by maintaining pillar sidewall integrity (Evermine)</td>
</tr>
<tr>
<td>Kuijpers (2004)</td>
<td>3 mm / 15 m section of pillar sidewall and hanging wall / 2 months / Serious stability problems</td>
<td>TSL provided excellent areal coverage by maintaining pillar sidewall and hanging wall integrity (Evermine)</td>
</tr>
<tr>
<td>Laurence (2004)</td>
<td>1 to over 5 mm / 20 m section of rib / 18 months / Rib in reasonable condition, but well cleated and friable with many cracks &gt; 5 mm</td>
<td>While flaking was evident in the adjacent non-sprayed areas, the roof/rib interface was generally intact in the sprayed area. Evermine appeared to prevent rib unravelling (Evermine)</td>
</tr>
<tr>
<td>Laurence (2004)</td>
<td>1 to over 5 mm / A corner of a heading / N.A. / Reasonably intact but many heavily cleated and friable with large cracks &gt; 5 mm</td>
<td>The result was inconclusive (Evermine)</td>
</tr>
<tr>
<td>Laurence (2004)</td>
<td>1 to over 5 mm / A section of two ribs / N.A. / Ribs in stressed condition</td>
<td>Evermine sprayed area experienced much less displacement and had much better conditions than the unsprayed area. The TSL reduced the depth of softening in the rib (Evermine)</td>
</tr>
<tr>
<td>Laurence (2004)</td>
<td>1 to over 5 mm / A section of belt road blockside / N.A. / Ribs in reasonable condition</td>
<td>Evermine covered blockside was intact until the long wall face approached closely (Evermine)</td>
</tr>
<tr>
<td>Laurence (2004)</td>
<td>1 to over 5 mm / A corner / N.A. / Intensely spalled and shattered</td>
<td>The result was inconclusive (Evermine)</td>
</tr>
<tr>
<td>Laurence (2004)</td>
<td>1 to over 5 mm / A goaf end chock / N.A. / N.A. /</td>
<td>The result was inconclusive (Evermine)</td>
</tr>
<tr>
<td>Hawker (2004)</td>
<td>5 mm / 3 m × 220 m longwall face plus 0.5 m on the roof for the lift-off process / N.A. / The coal seam spalls and weathers quickly /</td>
<td>The lift-off process was successful without personnel injury (Tekflex®)</td>
</tr>
</tbody>
</table>

2.5 Summary and conclusions

Roof stability has always been an issue in underground coal mining. In order to effectively support the roof of underground mines, it is important to understand roof failure mechanisms. Many roof failure modes have been observed, which include span
failure, skin failure, failure due to roof structure, mid-span shearing, shear failure, buckling and beam failure and guttering failure. Also, many roof support principles have been presented.

As a traditional roof support medium, steel mesh has been successfully used in combination with rock bolts in underground coal mines for a long time. Extensive experimental studies on the behaviour of steel mesh have been conducted. Many factors, such as steel wire diameter, bolt spacing, size of the loading area, loading plate orientation, bolt tension, load surface, bearing plate size and mesh size, have been shown to affect the behaviour of steel mesh. In spite of the successful use in underground strata control, steel mesh has some intrinsic disadvantages, such as it is a passive support, it is difficult to mechanise and it is vulnerable to corrosion. As such, an innovative roof support medium known as a TSL was proposed as a potential replacement. Since the aim of this research was to evaluate the potential of replacing steel mesh with TSLs for rapid roadway development in coal mines, shotcrete was not introduced in this study.

Thin spray-on liners, which are a relative new roof support technology, have many advantages over steel mesh. The primary merits of TSLs are the quick application rate, their positive support nature, and the ability to generate reaction force at small rock displacement and provide immediate support after excavation. TSLs have been widely studied using laboratory testing, numerical modelling and in situ testing. Many small and large scale laboratory tests have been conducted to determine the tensile, flexural, adhesion, shear, shear bond properties and load bearing ability of TSLs. Unlike tensile and flexural tests, there has been no standard test method for determining the other mechanical properties mentioned above. However, many test methods have been proposed, such as embedded dolly pull test, glued dolly pull test, core to core pull test and centre push test for the adhesion strength determination, the punch through tests for shear strength determination, and the DSS (double sided shear) test and DRP (double ring punch) test for shear bond strength determination. The substrate type, surface contaminants, substrate surface moisture, cure time and liner thickness have
all been found to have significant influence on TSL adhesion. In addition to small scale laboratory tests, large scale tests have also been conducted in the laboratory to study the load bearing capacity of TSLs. However, while some of the tests did not provide the critical parameters such as liner size and liner thickness, the liner size in most tests was not big enough and thus cannot closely represent the actual behaviour in situ. In addition to laboratory tests, numerical methods have been used to evaluate the potential of the TSL. FLAC$^{3D}$ appears to be a suitable modelling program. Many TSL field trails in underground coal mines have also been conducted, and the results indicated that the TSL can be successfully used for areal support.

In Chapter 3, the basic mechanical properties of sandstones, hydrostone plaster, TSLs and steel wire are determined by the use of laboratory tests. Determination of the mechanical properties of the materials is necessary to analyse the potential of TSLs in rock support and to provide the input parameters for the numerical models.
3 DETERMINING THE MECHANICAL PROPERTIES OF VARIOUS MATERIALS FOR USE IN NUMERICAL MODELLING

3.1 Introduction
A thin spray-on liner (TSL) is designed to bond onto the surface of the roadways of underground coal mines, which makes it a component of the composite with the substrate. In order to better understand the performance of thin spray-on liners (TSLs) in underground coal mine roof support, both large scale laboratory experiments and numerical modelling are covered in this thesis. Analysis of the results of laboratory tests and numerical simulations requires comprehensive data on the mechanical properties on the TSL and substrate materials. As such, a series of small scale laboratory tests were carried out to determine the tensile, flexural, adhesion and shear bond properties of prototype TSLs, and the uniaxial compressive strength, Young’s modulus, Poisson’s ratio and tensile strength of the substrates. Sandstone and hydrostone plaster were used as substrate materials.

The hydrostone plaster used was a mixture of gypsum plaster and Portland cement. It was reported that the UCS of gypsum plaster increases with increasing curing time (Bahorich 2012) and higher curing temperature produces greater UCS of a sand/plaster mixture (Yavuz & Fowell 2003). Therefore, in this chapter, the influence of different curing times and environments on the compressive and flexural properties of the hydrostone plaster was also investigated to ensure accurate analysis of the performance of TSLs.

In addition to the testing to determine the mechanical properties of the TSLs, laboratory experiments were also performed to determine the tensile strength, flexural strength and weld shear strength of the steel wire and associated welds that welded steel mesh, currently utilised in Australian underground coal mines, is comprised. The properties obtained contributed to the input parameters for the numerical modelling.
3.2 Determining the mechanical properties of hydrostone plaster and sandstone

In this study, sandstone and hydrostone plaster were selected as the substrates. Sandstone was used as it is a common substrate found in underground coal mines, while hydrostone plaster was used for its performance repeatability. Plaster, which is a polycrystalline material made of intricate gypsum needles, is often used as a rock mechanics modelling material (Coquard & Boistelle 1994) in the laboratory. It is manufactured by heating gypsum to about 150°C:

$$2CaSO_4 \cdot 2H_2O \xrightarrow{\text{Heat}} 2CaSO_4 \cdot \frac{1}{2}H_2O + 3H_2O$$

When the dry plaster powder is mixed with water, a chemical reaction of hydration of calcium sulphate hemihydrate occurs (Çolak 2006):

$$2CaSO_4 \cdot \frac{1}{2}H_2O + 3H_2O \xrightarrow{} 2CaSO_4 \cdot 2H_2O$$

Hydrostone plaster is a mixture of gypsum plaster and Portland cement. It has a reported UCS of around 69 MPa, whereas pure gypsum plaster has a UCS of only approximate 10 MPa (Bahorich 2012). Weaker materials often fail in the substrate rather than at the interface of the TSL and the substrate. Provided the appropriate ratios of powder and water are used, a consistent material which can used for modelling is produced, which facilitates repeatability of tests.

A variety of laboratory tests, which included the uniaxial compressive test, triaxial compressive test and Brazilian disc test, were conducted on hydrostone plaster and two types of sandstone. These tests were performed to provide the stress-strain behaviour and various parameters such as tensile strength, Young’s modulus, Poisson’s ratio, internal angle of friction and cohesion to be used in numerical modelling of the materials. As these tests were done in accordance with International Society for Rock Mechanics (ISRM) standards (Brown 1981), only a brief description of the test equipment and procedure is presented.
3.2.1 Uniaxial compressive tests

3.2.1.1 Preparation of samples

Hydrostone plaster and two types of sandstone cylinders were prepared for the uniaxial compressive test. Sandstone cylinders, 54 mm diameter, were cored from sandstone blocks. In order to avoid any possible damage to the core samples, the sandstone blocks were encapsulated in a concrete casting. The coring operation was conducted using a radial ram drilling machine. After coring, the sandstone cylinders were cut to around 140 mm in length using a diamond saw, which makes the height to diameter ratio of the sample approximately 2.6 : 1. The two end faces of each cylinder were then ground to be smooth and parallel with a lapping machine. The maximum deviation of end faces from parallelism did not exceed 0.05 degrees.

Compared to the sandstone samples, the preparation of plaster specimens was relatively simple. Plaster powder and water with a weight ratio of 3.5 : 1 was mixed till homogenous and then poured into a mould. The plaster sample was allowed to cure for one hour before it was taken out of the mould. As before, the plaster cylinders were polished to ensure smooth and parallel end faces. The plaster samples were 54 mm in diameter and 120 mm in length, a height to diameter ratio of 2.2 : 1. The height of the plaster samples were shorter than that of the sandstone samples, this was a result of the sandstone samples being prepared and tested prior to the preparation of plaster samples, when it was realized that there was only one mould (120 mm in height) to cast the plaster samples. It will be shown in equations (3.2) and (3.3) that decreasing sample height from 140 mm to 120 mm has negligible influence on the UCS of the samples.

For the purpose of this research, the deformability of the sandstone and plaster were also investigated using strain gauges. Two strain gauges were mounted on the specimen with one in the horizontal direction another in the vertical direction (Figure 3.1). Before the test, the sandstone samples were placed in a 45°C oven for three days
as they were saturated with water during the coring and cutting process. The plaster samples were allowed to cure in a room environment for three days.

![Sample with strain gauges](image)

**Figure 3.1 Sample with strain gauges**

### 3.2.1.2 Test procedure and results

A servo controlled machine was employed to carry out the tests, and displacement control model was applied during these tests. The stress can be calculated with:

\[
\sigma_a = \frac{P}{A}
\]

(3.1)

Where:

- \(\sigma_a\) = axial stress (MPa)
- \(P\) = the applied axial load (N)
- \(A\) = the initial cross-sectional area (mm²)

The Young’s modulus of the three materials were determined from the corresponding average slopes of the more-or-less straight line portion of the axial stress-axial strain curve, and the Poisson’s ratios by dividing the slope of axial stress-strain curve by the slope of diametric stress-strain curve (ISRM 1979). The average results for 11 samples are shown in Table 3.1. It is important to note that the UCS of the plaster was 38 MPa instead of the reported 69 MPa (USG 1999), which was because the plaster samples were not fully cured. This will be discussed in section 3.3.
Table 3.1 Mechanical properties of sandstones and hydrostone plaster from uniaxial tests

<table>
<thead>
<tr>
<th>Sample type</th>
<th>UCS/standard deviation (MPa)</th>
<th>Young’s modulus/standard deviation (GPa)</th>
<th>Poisson’s ratio/standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrostone plaster</td>
<td>38/1</td>
<td>29/1</td>
<td>0.31/0.02</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>24/3</td>
<td>28/2</td>
<td>0.23/0.03</td>
</tr>
<tr>
<td>Sandstone (f.g.)</td>
<td>33/3</td>
<td>35/4</td>
<td>0.29/0.04</td>
</tr>
</tbody>
</table>

Note: c.g. refers to coarse grain and f.g. refers to fine grain.

3.2.2 Triaxial compressive tests

The rocks in underground coal mines are generally under a triaxial stress state. It is common knowledge that the behaviour of rock in a confined situation is different from that of unconfined rock. The rock is stronger in terms of compressive strength when confined by surrounding rocks. In this study, triaxial compressive tests were conducted on plaster and two types of sandstone to obtain the internal angle of friction and cohesion of the materials.

3.2.2.1 Preparation of samples

The preparation procedure was the same as that of the uniaxial compressive test. The samples for this test were 54 mm in diameter and 120 mm in length. It is worthwhile to note that the height of the sandstone samples for the triaxial test were shorter than those in the uniaxial compressive test, due to the relatively short height of the triaxial cell. The effect of height/diameter ratio on the UCS of rock was studied by Protodyakonov in 1969 (cited in Singh & Ghose 2006) and it was reported as:

\[ \sigma_{c=2} = \frac{8 \sigma_{c>1}}{7 + 2 \frac{d}{H}} \]  \hspace{1cm} (3.2)

Where:
\( \sigma_{c>1} \) = the uniaxial compressive strength (MPa) of the sample with height/diameter ratio > 1

\( h \) = the height of the sample (mm)

\( d \) = the diameter of the sample (mm)

\( \sigma_{c=2} \) = calculated uniaxial compressive strength (MPa) of the sample with height/diameter ratio of 2

According to Equation 3.2, the UCS of the sample was increased by 1.4% when the height reduced from 140 mm to 120 mm, which was negligible. Thus, it can be considered that sandstone cylinders with the two different heights had similar UCS. This is also confirmed by ASTM 1986 in which:

\[
\sigma_{c=2} = \frac{\sigma_{c#2}}{0.88 + 0.24 \frac{d}{h}}
\]  

(3.3)

Where:

\( \sigma_{c#2} \) = the uniaxial compressive strength (MPa) of the sample with height/diameter ratio not equal to 2

\( h \) = the height of the sample (mm)

\( d \) = the diameter of the sample (mm)

\( \sigma_{c=2} \) = calculated uniaxial compressive strength (MPa) of the sample with height/diameter ratio of 2

According to this Equation, an increase of 1.5% in UCS was predicted when the height decreased from 140 mm to 120 mm, again not statistically significant. As before, the sandstone samples were placed in a 45°C oven for three days while the plaster samples were allowed to cure in a room environment for three days.

3.2.2.2 Test procedure and results

In the triaxial compressive test, the sample was firstly covered by a rubber membrane which was used to prevent oil from penetrating into sample, and then the specimen was loaded into the triaxial cell. A confining stress was applied to the sample by the use of a hydraulic pump, and the axial stress was applied via a ram. During the test,
the confining stress was increased incrementally from 1 MPa to 2 MPa and then 3 MPa. The detailed results are presented in Table 3.2. Figure 3.2 shows the Mohr circles of plaster and sandstone from the triaxial test.

Table 3.2 Triaxial strength of sandstones and hydrostone plaster

<table>
<thead>
<tr>
<th></th>
<th>Hydrostone plaster</th>
<th>Sandstone (c.g.)</th>
<th>Sandstone (f.g.)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Confining stress (MPa)</td>
<td>1  2  3</td>
<td>1  2  3</td>
<td>1  2  3</td>
</tr>
<tr>
<td>Triaxial strength (MPa)</td>
<td>45  51  56</td>
<td>29  36  38</td>
<td>40  44  50</td>
</tr>
<tr>
<td><strong>2</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Confining stress (MPa)</td>
<td>1  2  3</td>
<td>1  2  3</td>
<td>1  2  3</td>
</tr>
<tr>
<td>Triaxial strength (MPa)</td>
<td>46  50  56</td>
<td>30  36  38</td>
<td>40  44  49</td>
</tr>
<tr>
<td><strong>3</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Confining stress (MPa)</td>
<td>1  2  3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Triaxial strength (MPa)</td>
<td>44  52  54</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Average strength (MPa)</strong></td>
<td>45  51  55</td>
<td>30  36  38</td>
<td>40  44  50</td>
</tr>
<tr>
<td><strong>Standard deviation (MPa)</strong></td>
<td>1  1  1</td>
<td>1  0  0</td>
<td>0  0  1</td>
</tr>
</tbody>
</table>

The internal angle of friction, $\phi$, and cohesion, $C_{oh}$, of the materials may be calculated using the following formulas:

$$\phi = \arcsin\frac{m-1}{m+1}$$

$$C_{oh} = x\frac{1-\sin\phi}{2\cos\phi}$$

Where $m$ is the slope of the Coulomb strength envelope in the principal stresses plot, and $x$ is the value corresponding to the intercept of the strength envelope in axial stress axis. The results are shown in Table 3.3.
3.2.3 Brazilian disc test

The tensile strength of rocks is normally much less than its compressive strength, being around 1/8-1/10. As tensile strength of rocks is a basic parameter in numerical modelling, it is necessary to determine the tensile strength of materials. There are three methods available to determine the tensile strength of rocks. One is a direct method, the others are indirect methods which include the Brazilian disc test and the point load test. Even though the direct method is more accurate, indirect methods are more often

<table>
<thead>
<tr>
<th></th>
<th>Plaster</th>
<th>Sandstone (c.g.)</th>
<th>Sandstone (f.g.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal angle of friction (°)</td>
<td>43</td>
<td>38</td>
<td>41</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>4.6</td>
<td>3.9</td>
<td>4.6</td>
</tr>
</tbody>
</table>

Figure 3.2 Mohr circle of plaster and sandstone from triaxial test
utilized as they are simple to perform and they are not as time consuming as the direct method (Hematian 1994). In this study, the Brazilian disc test was performed on the plaster and two types of sandstone to determine their tensile strength.

3.2.3.1 Preparation of samples
Preparation of samples was similar to that for the uniaxial compressive test, except that the 54 mm cores were cut into squat cylinders with a thickness of around 27 mm. Before the test, the sandstone samples were placed in a 45°C oven for three days while the plaster samples were allowed to cure in the room environment for two weeks.

3.2.3.2 Test procedure and results
All together 30 samples were subjected to the Brazilian disc test. The indirect tensile strength of the samples were calculated using the following equation, the results are presented in Table 3.4.

\[ \sigma_t = \frac{2P_f}{\pi td} \]  

(3.6)

Where:

\( \sigma_t \) = indirect tensile strength (MPa)

\( P_f \) = failure load (N)

\( t \) = thickness of the sample (mm)

\( d \) = diameter of the sample (mm)

Table 3.4 Tensile strength of sandstones and hydrostone plaster from Brazilian disc tests

<table>
<thead>
<tr>
<th>Sample type</th>
<th>Tensile strength (MPa)</th>
<th>Standard deviation (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrostone plaster</td>
<td>6.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>2.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Sandstone (f.g.)</td>
<td>3.1</td>
<td>0.2</td>
</tr>
</tbody>
</table>
Comparing Table 3.1 with Table 3.4 we find that while the tensile strength of sandstones is approximately 1/10 of the UCS, the tensile strength of plaster is around 1/6 of the UCS. The reason is that the plaster samples for the uniaxial compressive test were cured for 3 days in room condition before testing while the curing time of the samples for the Brazilian disc test was 2 weeks. This suggests that the preparation of cured plaster varied with the time of cure. This will be investigated in the following section, as will be the effect of the curing environment.

Table 3.5 provides a summary of the results obtained from the various tests.

<table>
<thead>
<tr>
<th></th>
<th>Hydrostone plaster</th>
<th>Sandstone (c.g.)</th>
<th>Sandstone (f.g.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS (MPa)</td>
<td>38</td>
<td>24</td>
<td>33</td>
</tr>
<tr>
<td>Young’s modulus (GPa)</td>
<td>29</td>
<td>28</td>
<td>35</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.31</td>
<td>0.23</td>
<td>0.29</td>
</tr>
<tr>
<td>Internal angle of friction (°)</td>
<td>43</td>
<td>38</td>
<td>41</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>4.6</td>
<td>3.9</td>
<td>4.6</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>6.6</td>
<td>2.3</td>
<td>3.1</td>
</tr>
</tbody>
</table>

3.3 **Effect of different curing conditions on the mechanical properties of hydrostone plaster**

The reported UCS of hydrostone was 69 MPa, Table 3.5, however, shows a UCS of only 38 MPa. It was found from another series of tests that the mechanical properties of hydrostone plaster are not stable when the curing conditions change, all other factors, such as mix ratio, being constant. With respect to the use of the hydrostone plaster, it was thus necessary to determine how long and under what conditions the hydrostone plaster sample should cure so that consistency of samples were maintained. As such, a series of tests were conducted to evaluate the effect of different curing times and environments on the compressive and flexural properties of the hydrostone plaster.
3.3.1 Methodology

Uniaxial compressive strength and flexural strength were selected as the two properties to be determined as a function of various curing times and environments. The curing times were one day, three days, five days, one week, two weeks, three weeks, four weeks, eight weeks and twelve weeks. The curing environments were room conditions and a 45°C oven.

3.3.1.1 Uniaxial compressive test

The sample preparation and test procedures were identical to that in section 3.2. The samples were divided into 18 groups and experienced various curing conditions as stated above.

3.3.1.2 Three-point bending test

Hydrostone powder and water were mixed as before to cast rectangular plaster blocks with dimensions of 160 mm × 40 mm × 40 mm. While one group of samples was stored in a 45°C oven the other group was kept in a room environment for the predetermined curing times. An Instron servo-hydraulic testing machine applied load at a constant rate of 0.1 mm/min. The plaster block was supported by two steel rollers lying at the same distance from the centre, with a support span of 120 mm, Figure 3.3 shows the schematic of the three-point bending test. The load versus displacement behaviour of the plaster was recorded during the test.

Figure 3.3 Schematic of three-point bending test
3.3.2 Results

The average test results at each curing time and both curing environments for the UCS tests are illustrated in Figure 3.4. As expected the oven cured plaster samples exhibited a more rapid increase in strength than the corresponding room cured samples, but possibly not so intuitive was that the peak strength of the oven cured samples was greater than the room cured samples at 76 MPa and 62 MPa respectively. It is evident that the oven cured samples underwent a rapid increase in UCS in the first three days from 39 MPa to 63 MPa, termed the rapid strengthening stage, but there was not much change in the UCS of the room cured samples, slow strengthening stage. The UCS of both the room and oven samples reached a peak after two weeks, the oven having undergone slow strengthening from day three to week two, while the room samples underwent rapid strengthening from week one to week two. After week two all samples remained stable during the period from week two to somewhere between weeks four and eight, but then experienced a slight decrease, a weakening stage. This suggests that samples should not be used for modelling after around six weeks of curing. No matter what the curing conditions all samples exhibited brittle failure in compression.

Figure 3.4 Uniaxial compressive test results
Figure 3.5 illustrates the typical load versus displacement curve of a 3-point bending test on the plaster. The slope of the load versus displacement curve can be calculated from the following Equation:

$$k = \frac{l_p - l_{50}}{d_p - d_{50}}$$  \hspace{1cm} (3.7)

Where $k$ is the slope, $l_p$ is the peak load and $d_p$ is the corresponding displacement, $l_{50}$ is 50% of the peak load and $d_{50}$ is the displacement at $l_{50}$.

Figure 3.5 Typical load versus displacement curve for a three-point bending test of plaster

The flexural strength of the plaster is calculated using the following Equation:

$$\sigma_f = \frac{3P_f l}{2bt^2}$$  \hspace{1cm} (3.8)

Where:

$\sigma_f$ = flexural strength of the sample (MPa)

$P_f$ = load at failure (N)

$l$ = span length (mm)

$b$ = sample width (mm)

$t$ = sample thickness (mm)
The average results of the three-point bending tests are shown in Figure 3.6. Generally the displacement at failure of the plaster increased as its flexural strength rose. Plasters cured in the oven had greater flexural strength and displacement at failure than the room cured samples, the peak flexural strength of the oven group was 16 MPa with a corresponding displacement at failure of 0.4 mm while the room cured plasters were 12 MPa and 0.3 mm respectively.

For the oven group, the flexural strength of the plaster increased significantly from day one at 7 MPa to day three at 13 MPa, from then till week four it held at around 13 MPa. The peak flexural strength of 16 MPa was found at week eight, and after that the plaster experienced a significant decrease in flexural strength. For the room cured plasters, the flexural strength was stable at about 7 MPa during the first week and then started to grow, reaching a peak of 12 MPa in week eight. As before the plaster became weaker from then on. It is important to note that although different curing times and environments affected the flexural properties of the plaster, as with the compression test, the material still experienced brittle failure.

As with the compression sample, the change of flexural strength of the plasters can be divided into different stages, Figure 3.6. Unlike the compression samples which had four stages, the room cured samples exhibited only three stages: a stable stage, a slow strengthening stage and a weakening stage. The stable stage occurred in the first week of curing, followed by the slow strengthening stage which lasted to the eighth week and then the weakening stage. For the oven cured samples, as with the compression samples, there were four stages: a fast strengthening stage for the first three days, a stable stage which lasted to week four, a slow strengthening stage in the following four weeks and finally a weakening stage. The two types of sample experienced different stages of flexural strength variation, which was probably induced by different water evaporation processes with respect to the two kinds of curing environment.
3.3.3 Discussion

Laboratory tests were performed to investigate the effect of different curing times and environments on the mechanical properties of the hydrostone plaster which is frequently used in the geomechanics laboratory of University of Wollongong. The test results indicated that the oven cured hydrostone samples not only achieved a significant increase in both the UCS and flexural strength much quicker, but they were also stronger than the room cured samples. This was due to the higher temperature in the oven helping to evaporate the excess water in the hydrostone. Coquard and Boistelle (1994) stated that water molecules may significantly weaken the bonds between the crystals of the plaster and eventually result in a drastic decrease in its strength. Specifically, while it took two weeks for the ‘room samples’ to achieve a significant increase in mechanical resistance the ‘oven samples’ increased dramatically in both UCS and flexural strength in the first three days. The peak UCS of the oven samples was 75 MPa, which was about 20% greater than that of room samples, and the peak flexural strengths of the two groups were around 16 MPa and 12 MPa respectively.

Figure 3.6 Flexural properties of plasters
No matter under which curing environments, the hydrostone initially increased in UCS and flexural strength over a limited time, reached a stable stage for a number of weeks and then experienced a decrease in strength owing to degradation of the cured sample over extended time periods. The curing times to reach the stable stage of the mechanical properties of hydrostone samples were different with respect to different curing conditions. It is important to make sure the hydrostone samples are in the stable stage when they are employed as a rock simulation material, otherwise sub-optimum results may be obtained.

3.4 Determining the mechanical properties of a thin spray-on liner

The tensile strength, flexural strength, adhesive strength and shear bond strength of prototype thin spray-on liners (TSLs) were determined. Unlike tensile strength and flexural strength there is no standard test method available for adhesion strength and shear bond strength. In this chapter, the tensile and flexural properties of the TSL were assessed using the corresponding Australian Standards. The adhesive strength test was conducted following the method developed by Tannant and Ozturk (2003) and a modified version of the test method proposed by Saydam et al (2003) was employed to study the shear bond behaviour of the thin spray-on liner (TSL).

Three types of resin (A, B and C) were selected as the base polyester for the prototype TSLs. Resins A and B were readily available commodities. Resin C was a prototype base polyester for a TSL under development. Resin A was a polyester based polymer formulation with a cross linking monomer. It comprised 70% polyester and 30% monomer. Resin B was a general purpose laminating resin. It was manufactured using a medium reactivity, rigid orthophthalic polyester base resin. Resin C comprised approximately 50% polyester resin and approximately 50% fillers consisting of CaCo3, TiO2 and aluminum trihydrate. When mixed with an appropriate curative, the resins become polymers A, B and C respectively.
3.4.1 Tensile properties investigation

Tensile tests on polymer A, fibre reinforced polymer (FRP) A, polymer B and FRP B, and polymer C and two types of FRP C were conducted in accordance with Australia Standard 1145.2-2001, Determination of tensile properties of plastic materials Part 2: Test conditions for moulding and extrusion plastics, and Australia Standard 1145.4-2001, Determination of tensile properties of plastic materials Part 4: Test conditions for isotropic and orthotropic fibre-reinforced plastic composites respectively. The shape and dimensions of AS 1145.2 Type 1A test specimens were suitable for testing each polymer, while Type 2 test specimens of AS 1145.4 were used for the FRP. Two sets of polymer A samples were subject to tensile testing, one set of dog-bone shaped polymer A samples and one set of rectangular polymer A samples reinforced with 3 layers of fibre sheet (FRPA_3S). Three groups of polymer B sample were tested, which were polymer B, polymer B reinforced with 2 layers of fibre sheet (FRPB_2S) and polymer B reinforced with 3 layers of fibre sheet (FRPB_3S). Tensile tests on polymer C, polymer C reinforced with 3 layers of fibre sheet (FRPC_3S) and polymer C reinforced with bar-chip fibre (FRPC) were also performed in accordance with the above mentioned standards. The component weights of each FRP are presented in Table 3.6.

<table>
<thead>
<tr>
<th></th>
<th>FRPA_3S</th>
<th>FRPB_2S</th>
<th>FRPB_3S</th>
<th>FRPC_3S</th>
<th>FRPC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fibre weight (g)</td>
<td>8.6</td>
<td>5.7</td>
<td>8.6</td>
<td>7.3</td>
<td>2.5</td>
</tr>
<tr>
<td>Polymer weight (g)</td>
<td>37.6</td>
<td>26</td>
<td>26.5</td>
<td>39.1</td>
<td>55</td>
</tr>
<tr>
<td>Fibre weight percentage</td>
<td>18.6%</td>
<td>18%</td>
<td>24.5%</td>
<td>15.8%</td>
<td>4.3%</td>
</tr>
</tbody>
</table>

3.4.1.1 Preparation of samples

The dimensions of the dog-bone and rectangular samples for the unreinforced and fibre reinforced polymer as recommended by the Australian standards are shown in Figure 3.7 (a) and (b) respectively.

82
Polymer A and B samples were prepared by gently pouring the viscous polymer into a latex mould (Figure 3.8a) and driving out the air bubbles after the completion of pouring by the use of a pin. The samples were removed from the mould after the polymer set, and then any burrs were trimmed off. The samples were then placed in a 60 degree oven for 24 hours to achieve full cure.

(a) Dimensions of dog-bone shaped sample (all dimensions in mm)

(b) Dimensions of rectangular sample (all dimensions in mm)

Figure 3.7 Dimensions of the samples

Figure 3.8 Preparation of polymers A and B samples for tensile testing
Preparation of the FRP A and B was similar to the unreinforced samples except that a different mould was used and glass fibre sheets were embedded into the polymer. Taking the preparation of polymer B reinforced with 2 layers of fibre as an example, the glass fibre sheets were cut into size and the polymer was divided into three equal batches. The mould was placed on a flat surface and the first batch of polymer was poured into the mould and distributed evenly by the use of a spatula. Then the first glass fibre sheet was placed onto the uncured polymer and gently rolled till the fibre sheet was completely saturated with polymer. After that, the second batch of polymer was poured, followed by the application of another fibre sheet. Finally, the last batch was poured and evenly distributed. The sample was allowed to set in the mould and then cured in a 60 degree oven for 24 hours. The FRP B plate was then machined into the desired size. The other FRP samples were prepared in a similar way.

![Figure 3.9 Preparation of FR sample for tensile testing](image)

Polymer C samples were prepared by firstly pouring the polymer into the mould, as the polymer was too viscous to prepare in the smaller moulds, and then cutting and machining the polymer plate to the shape and size as indicated in Figure 3.7 (a) after it cured. FRPC_3S was prepared the same way as FRPA_3S. Preparation of FRPC was slightly more complex. The polymer was divided into two equal batches. After the first batch was poured into the mould and evenly distributed, bar-chip fibre (48 mm in length, 640 MPa in tensile strength and 10 GPa in Young’s Modulus) was evenly sprayed onto the surface of the polymer. Then the other batch was applied. The FRP plate was machined to the desired size after curing.
In order to evaluate the deformation properties of the TSL materials, strain gauges were fixed to some of the samples to monitor the strain in the longitudinal and transverse direction.

3.4.1.2 Test set-up and procedure

An Instron testing machine was used to perform the tensile tests, the test set-up is illustrated in Figure 3.10. In this test, the sample was held by two grips at the ends. The tightening of grips should neither be too strong to result in pre-mature failure nor too weak to lead to slip of the specimen during the test. Moreover, it was important to guarantee the pull force is parallel to the longitudinal axis of the sample otherwise an inaccurate result would be produced due to eccentric loading. The tensile test was done under displacement control mode. The loading rates for unreinforced polymer and FR polymer were different, being 1 mm/min and 2 mm/min respectively, as recommended by the standards. As is shown in Figure 3.10 the bottom grip was fixed and the top grip raised at the pre-determined rate till the sample breaks. The load and extension were recorded during the test process. The dog-bone shaped samples should break at the narrow section, the test was considered invalid if failure occurred at any other location.

![Figure 3.10 Tensile test set-up](image)
3.4.1.3 Results and discussion

The tensile strength was calculated using the following equation:

\[
\sigma_t = \frac{P_f}{A}
\]  

(3.9)

Where:

- \(\sigma_t\) = tensile strength (MPa)
- \(P_f\) = load at failure (N)
- \(A\) = original cross-sectional area of the specimen at the narrow section (mm\(^2\))

The detailed tensile test results of all the samples are shown in Figure 3.11.

![Tensile strength of TSLs](image)

**Figure 3.11 Tensile strength of TSLs**

The Young’s modulus and Poisson’s ratio of the TSLs were calculated using the same method as described in Section 3.2.1.2. Figure 3.12 shows the stress versus strain curves for polymer A and polymer A reinforced with 3 layers of fibre sheet (FRPA_3S). The different colours of the curves represented different samples.
Figure 3.12 Stress versus strain curves of polymer A and polymer A reinforced with 3 layers of fibre sheet

Table 3.7 presents the average results of the tests. It is obvious from the Table that the fibre contributed to an increase in the tensile strength of the TSLs. While polymer A failed at a stress of 14 MPa, the failure stress of polymer A reinforced with 3 layers of fibre sheet (FRPA_3S) was 55 MPa which is around four times the strength of polymer A. When it came to polymer B, an increase of 212% in tensile strength was achieved when the polymer was reinforced with 2 layers of glass fibre sheet (FRPB_2S), and the value reached 271% when the polymer was reinforced with 3 layers of glass fibre (FRPB_3S). Compared to polymer A and B, the application of bar-chip fibres did not
significantly increase the tensile strength of polymer C as had been expected, with the reinforcement being 17%. This was because polymers A and B were reinforced with more fibre (18.6% and 24.5% respectively) than polymer C (4.3%) as shown in Table 3.6. When polymer C was reinforced with 3 layers of glass fibre sheet (FRPC_3S), its tensile strength increased significantly to 42 MPa.

Table 3.7 Tensile properties and deformability of TSLs

<table>
<thead>
<tr>
<th></th>
<th>Tensile strength</th>
<th>Young’s modulus</th>
<th>Poisson’s ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>/standard deviation</td>
<td>/standard deviation</td>
<td>/standard deviation</td>
</tr>
<tr>
<td></td>
<td>(MPa)</td>
<td>(GPa)</td>
<td>(GPa)</td>
</tr>
<tr>
<td>Polymer A</td>
<td>14/1</td>
<td>2/0.2</td>
<td>0.36/0.02</td>
</tr>
<tr>
<td>FRPA_3S</td>
<td>55/2</td>
<td>9/0.5</td>
<td>0.33/0.02</td>
</tr>
<tr>
<td>Polymer B</td>
<td>17/1</td>
<td>3/0.5</td>
<td>0.34/0.03</td>
</tr>
<tr>
<td>FRPB_2S</td>
<td>53/1</td>
<td>5/0.2</td>
<td>0.38/0.01</td>
</tr>
<tr>
<td>FRPB_3S</td>
<td>63/2</td>
<td>9/1</td>
<td>0.32/0.01</td>
</tr>
<tr>
<td>Polymer C</td>
<td>12/1</td>
<td>1/0.02</td>
<td>0.35/0.02</td>
</tr>
<tr>
<td>FRPC</td>
<td>14/1</td>
<td>2/0.3</td>
<td>0.38/0.03</td>
</tr>
<tr>
<td>FRPC_3S</td>
<td>42/4</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

The application of the fibre reinforcement not only increased the tensile strength of the polymers, but also enhanced the elastic modulus. Specifically, while the elastic modulus of polymer A was 2 GPa, that of the FRPA_3S reached 9 GPa. An increase of 6 GPa in elastic modulus was achieved when polymer B was reinforced with 3 layers of fibre sheet. With respect to polymer C, the elastic modulus increased from 1 GPa to 2 GPa when the bar-chip fibre was incorporated. It is shown in Table 3.7 that the stiffness of FRPB_3S was greater than that of FRPB_2S. It is reasonable to envisage that increasing the content of the bar-chip fibre would produce a stiffer liner.

The increase in stiffness of the TSLs as a result of the application of the fibre is beneficial to roof support in underground coal mines in terms of preventing rock displacement. Under the same rock loading, a stiffer TSL is expected to have smaller deformation, which helps to preserve the inherent strength of the rock surrounding the
excavation surface. Fibre reinforcement had little if any effect on the Poisson’s ratio of the TSLs.

3.4.2 Flexural properties investigation

Flexural properties are some of the most important mechanical properties of TSLs. The flexural properties were investigated by conducting three-point bending tests according to Australia Standard 2132-1978, Determination of the flexural properties of plastics.

3.4.2.1 Sample preparation

As suggested by the standard mentioned above, the sample should be rectangular in shape with the dimensions of 100 mm × 10 mm × 4 mm for unreinforced polymer and 100 mm × 15 mm × 4 mm for FRP. Since there was no mould which was exactly the same size as suggested by the standard, the samples were prepared by firstly casting polymer and FRP plates and then machining them into beams of the required size. The preparation of the samples for the three-point bending test was almost identical to that of the samples for the tensile test except that a different mould (Figure 3.13) was used and the sample sizes were different.

Figure 3.13 Mould for preparation of samples for 3-point bending test

Two sets of polymer A samples were prepared for testing, one set of polymer A samples and one set of polymer A samples reinforced with 3 layers of fibre sheet (FRPA_3S). Three groups of polymer B sample were prepared, which were polymer
B, polymer B reinforced with 2 layers of fibre sheet (FRPB_2S) and polymer B reinforced with 3 layers of fibre sheet (FRPB_3S). Polymer C, FRP C (FRPC) and polymer C reinforced with 3 layers of fibre sheet (FRPC_3S) were also prepared for the three-point bending test.

3.4.2.2 Test set-up and procedure
Tests were conducted by the use of an Instron testing machine. Displacement control mode was selected. The loading rate was 2 mm/min for unreinforced polymer. As for FRP, the loading rate was 2 mm/min till the sample yielded and then increased to 4 mm/min. This was because the FRP had significant post failure behaviour, a typical test would take more than 10 minutes to complete if a loading rate of 2 mm/min was applied during the whole test process. The test set-up is shown in Figure 3.14. In this test, the specimen was supported by two rollers with a span of 70 mm, and the load was applied at the centre of the specimen by another downward moving roller. Load versus deflection relationship was recorded during the test.

![Figure 3.14 Flexural test set-up](image)

3.4.2.3 Results and discussion
The flexural strength can be calculated using the following Equation:

\[
\sigma_f = \frac{3P_{ps}l}{2bt^2}
\]  

(3.10)
Where:

\[ \sigma_f = \text{flexural strength (MPa)} \]

\[ P_p = \text{peak load (N)} \]

\[ l = \text{the span length (mm)} \]

\[ b = \text{the width of the test specimen (mm)} \]

\[ t = \text{the thickness of the test section (mm)} \]

The detailed flexural strength of each sample was plotted in Figure 3.15 and the average results are illustrated in Table 3.8. It is obvious from both the figure and the table that the fibre reinforcement contributed to an increase in the flexural strength. Specifically, polymer A had a flexural strength of 36.7 MPa, an increase of 145% was achieved when it was reinforced with 3 layers of fibre sheet. The flexural strengths of polymer B, FRPB_2S and FRPB_3S were 66.6 MPa, 71.7 MPa and 91.5 MPa respectively. The FRPC was slightly stronger than the unreinforced polymer C, with a flexural strength of 22.4 MPa and 22.0 MPa respectively. Likewise, this was because the bar-chip fibre embedded in polymer C was not as much as the glass fibre in the other two polymers, as shown in Table 3.6. When Polymer C was reinforced with more fibre (FRPC_3S), its flexural strength increased significantly to 58 MPa.
Table 3.8 Flexural strength and modulus of TSLs

<table>
<thead>
<tr>
<th></th>
<th>Flexural strength (MPa)</th>
<th>Flexural modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polymer A</td>
<td>36.7</td>
<td>1.2</td>
</tr>
<tr>
<td>FRPA_3S</td>
<td>89.8</td>
<td>2.3</td>
</tr>
<tr>
<td>Polymer B</td>
<td>66.6</td>
<td>2.9</td>
</tr>
<tr>
<td>FRPB_2S</td>
<td>71.7</td>
<td>3.4</td>
</tr>
<tr>
<td>FRPB_3S</td>
<td>91.5</td>
<td>3.8</td>
</tr>
<tr>
<td>Polymer C</td>
<td>22.0</td>
<td>1.0</td>
</tr>
<tr>
<td>FRPC</td>
<td>22.4</td>
<td>1.6</td>
</tr>
<tr>
<td>FRPC_3S</td>
<td>58</td>
<td>2.2</td>
</tr>
</tbody>
</table>

Typical load versus deflection curves of polymer A and FRPA_3S are shown in Figure 3.16(a). The peak load of polymer A was 51 N. As expected, the peak load of FRPA_3S was much higher, being about 275 N. The application of the 3 layers of fibre sheet contributed to an increase of 439% in the peak load. Moreover, it also changed the failure mode of the sample. While polymer A broke in a sudden way without any post failure behaviour, FRPA_3S firstly experienced a sharp fall in load bearing capacity from the peak of 275 N to approximately 162 N, and then underwent a slow weakening process. At the peak load, the polymer at the central bottom of the sample fractured followed by a reduction in the load resistance. And then the fibre and the bond of the fibre to the polymer started to generate resistance. It is worthwhile to note that the FRP was still able to bear a load of 27 N at the end of the test. The brittle failure of polymer A and plastic failure of FRPA_3S are described in Figure 3.17. It can be seen from this figure that while polymer A ruptured completely the FRPA_3S failed without breaking into two parts due to the fibre reinforcement. The significant post failure behaviour of the FRP is desirable for rock support in underground coal mines as it is able to provide a warning before it fails completely.
Figure 3.16 Typical load-deflection behaviours of TSLs
Figure 3.17 Failure of polymer A and polymer A reinforced with 3 layers of fibre sheet

It is apparent from Figure 3.16(b) and (c) that polymer B and C experienced similar trend as polymer A. Specifically, both polymers B and C failed in a brittle way while their fibre reinforced mediums showed ductile failure. Note that the post failure behaviour of FRP A (FRPA) and FRP B (FRPB) was slightly different from that of FRP C (FRPC). Specifically, after the peak load both FRPA and FRPB underwent a sharp fall to almost half of their peak loads, which was then followed by a slow softening process. The FRPC was still able to hold approximately 75% of the peak load after the first significant fall in load, and then the load decreased along a couple of parabolas. At the end of the test, the FRPC can still resist 50% of the peak load while the corresponding value for the other two FRP was approximately 10%. This was attributed to the difference in fibre configuration.

The flexural modulus, which is the ratio of stress to strain in flexural deformation, can be calculated using the following Equation:

\[ E_f = \frac{FLl^3}{4Ybt^2} \]  

(3.11)

Where:

\( E_f \) = the flexural modulus (MPa)

\( l \) = the span length (mm)

\( b \) = the width of the test specimen (mm)
\( t \) = the thickness of the test specimen (mm)

\( F \) = the force at a chosen point on the initial linear portion of the force-deflection curve (N)

\( Y \) = the deflection corresponding to force \( F \) (mm)

Figure 3.18 shows the calculated flexural modulus of each tested sample and the average flexural modulus of the TSLs are presented in Table 3.8. As expected, the fibre reinforcement not only increased the flexural strength of the polymer but also improved the flexural modulus of the polymer. The flexural modulus of polymer A was increased by 92% when reinforced with 3 layers of fibre. An increase of 17% and 31% in flexural modulus was achieved by polymer B when reinforced with 2 layers of fibre sheet and 3 layers of fibre sheet respectively. Likewise, the flexural modulus of FRPC was greater than that of the unreinforced polymer C, being 1.6 GPa and 1.0 GPa respectively. FRPC_3S was reinforced with more amount of fibre than FRPC. As expected, it had a greater flexural modulus of 2.2 GPa which was increased by 38% compared with FRPC. The greater the flexural modulus the more difficult to deform the TSL, which helps to prevent the rock mass from dilating and deteriorating and eventually preserve the inherent strength of the strata.

![Figure 3.18 Flexural modulus of TSLs](image-url)
3.4.3 Adhesion properties investigation

One of the advantages of TSLs over steel mesh in rock support is that TSLs are able to be bonded to the rock surface, which enables the TSLs to generate resistance even if small rock movement occurs. This benefit exists only if TSLs are tightly bonded to the rock surface. Thus the adhesion of TSLs plays an important role in rock support. The adhesion strength of TSLs has been studied by many researchers as presented in chapter two. This section attempts to determine the adhesion strengths of two types of polymer (A and B). Note that the adhesion strength of polymer C was not tested due to material scarcity. Although a standard test method for adhesion strength of the TSL is not available, many approaches (Spearing et al. 2004; Archibald 2004; Tannant & Ozturk 2004) have been proposed. In this study, the adhesion strength tests were conducted following the test method developed by Tanant and Ozturk (2004). As moisture is frequently encountered in underground coal mines, both dry and wet rock substrate surfaces were investigated.

3.4.3.1 Test fixture and sample assembly

The schematic of the adhesion test is described in Figure 3.19. A 5 mm thick TSL skin was bonded to a cylinder substrate, sandstone, with a diameter of 54 mm and height of 35 mm. A 38 mm diameter cylindrical dolly was glued normal to the surface of the TSL with an epoxy and a 5 mm deep kerf was cut into the TSL layer. The sample was clamped in an Instron testing machine during the test. The loading was applied by pulling the dolly upwards at a constant rate. Ideally, the failure occurs at the interface between the TSL and the substrate.
3.4.3.2 Preparation of samples

The procedures of sample preparation were as follows:

(i) The first step was to prepare the sandstone substrate. A sandstone block (Figure 3.20a) was selected and tightly fixed to the base of a coring machine (Figure 3.20b). Sandstone cylinders (Figure 3.20c) were then cored from the block, which in turn were cut to be 35 mm in thickness (Figure 3.20d). The cylinders were divided into two groups, one group (dry samples) was kept in the room environment and the other group (wet samples) was saturated in water for 24 hours.

(ii) The cylinders were then wrapped with two layers of para film (Figure 3.20e), which was to prevent the polymer flowing onto the periphery surface during the pouring process.

(iii) Another layer of plastic film was wrapped along the periphery of the cylinder (Figure 3.20f), and its top was 5 mm higher than the top surface of the cylinder.

Figure 3.19 Schematic of adhesion test
Polymer was then poured onto the top surface of the sandstone cylinder to create a 5 mm thick layer (Figure 3.21a). The sample was then left to cure in the room environment.

The aluminium dolly was glued onto the centre of the surface of the polymer with epoxy resin (Figure 3.21b). The sample was then left for 3 days before over-coring to allow the bond at the epoxy–dolly interface to develop.

A 5 mm deep groove was cut into the polymer along the periphery of the dolly using a 38 mm coring bit (Figure 3.21d). As such, the test area was equal to the area of the bottom surface of the dolly, which ensured consistency of loading area.
3.4.3.3 Test set-up and procedure

As shown in Figure 3.22, an Instron hydraulic testing machine was used to conduct the adhesion tests. During the test, the sample was placed in a metal tube which was restricted in all translational and rotational movements. The load was applied by pulling a bolt which was connected to the dolly with an internal thread. Displacement mode was selected and a constant rate of 0.5 mm/min was applied throughout the test. Load and displacement were recorded during the test.
3.4.3.4 Calculations

Ideally, de-bonding at the interface of the sandstone and polymer will occur as the load increases. The adhesion strength can be calculated using the following Equation:

$$\sigma_{ad} = \frac{P}{A_i}$$  \hspace{1cm} (3.12)

Where:

$\sigma_{ad}$ = the adhesion strength (MPa)

$P$ = the load at failure (N)

$A_i$ = the area of the interface (mm$^2$)

3.4.3.5 Failure mode

Three failure modes in the adhesion test were observed in a previous study (Ozturk & Tannant 2004): adhesive at the interface which is characterized by 0-33% substrate left on liner, combination which refers to 34-66% substrate left on liner, and cohesive in the substrate which features 67-100% substrate left on the liner. The failure of the sample depends on many factors such as the tensile strength of the substrate, the tensile bond of the polymer to the substrate, the bond of the epoxy to the polymer and the bond of the epoxy to the aluminium dolly. Correspondingly, the failure may occur in the sandstone, at the interface between the sandstone and the polymer, at the interface between the epoxy and the polymer and at the interface between the epoxy and the dolly. Ideally, the failure occurs in the second mode mentioned above.

As the bond of the epoxy to the polymer and the dolly were strong enough bond failure did not occur during the test. Since the strength of the sandstone was not as strong as expected, almost all of the failure occurred in the sandstone (Figure 3.23a). Partial failure at the sandstone-polymer interface (Figure 3.23b) was also discovered during the test.
3.4.3.6 Results and discussion

As mentioned above, only one sample experienced partial failure at the sandstone-polymer interface, all of the other samples failed in the sandstone. Thus, it was not possible to accurately calculate the adhesion strength of the polymer to the sandstone, however, it can be concluded that the adhesion strength was greater than the tensile strength of the sandstone. According to Table 3.4, the tensile strength of the sandstone was 3.1 MPa. It is possible that failure could occur at the interface between the polymer and substrate if a relatively stronger rock (such as granite) was used, however, it should be noted that the surface of the roadway in underground coal mines is usually soft rock.

The failure loads of polymer A and B samples are illustrated in Table 3.9 and Table 3.10 respectively. It is obvious that the failure loads for dry samples were generally greater than that for the wet samples. Note that failure load of A_wet_1 was much greater than the other two wet samples, this was because it was stored in a 50°C oven for 24 hours. Thus, it was not categorized into the wet group when calculating the average failure load. While dry polymer A samples failed at 2.2 kN the wet polymer A samples failed at approximately 1.4 kN. This was in agreement with the study of Dube and Singh (1969) in which dry sandstone was reported to have greater strength than sandstone saturated with water. The average failure loads for dry and wet polymer B samples were 1.7 kN and 0.7 kN respectively. Both dry and wet group polymer B samples failed at a lower load than their corresponding polymer A samples. This was...
possible because the sandstone substrate bonded to polymer B was weaker than that bonded to polymer A. As mentioned in section 3.2, two types of sandstone were used in this study.

Table 3.9 Failure loads of dry and wet polymer A samples during the test

<table>
<thead>
<tr>
<th></th>
<th>Dry_1</th>
<th>Dry_2</th>
<th>Dry_3</th>
<th>Dry_4</th>
<th>Wet_1</th>
<th>Wet_2</th>
<th>Wet_3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure load (kN)</td>
<td>2.4</td>
<td>2.2</td>
<td>2</td>
<td>2.2</td>
<td>2</td>
<td>1.5</td>
<td>1.3</td>
</tr>
<tr>
<td>Average failure load (kN)</td>
<td></td>
<td>2.2</td>
<td></td>
<td></td>
<td></td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>Standard deviation (kN)</td>
<td></td>
<td>0.2</td>
<td></td>
<td></td>
<td>0.2</td>
<td>0.1</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.10 Failure loads of dry and wet polymer B samples during the test

<table>
<thead>
<tr>
<th></th>
<th>Dry_1</th>
<th>Dry_2</th>
<th>Dry_3</th>
<th>Wet_1</th>
<th>Wet_2</th>
<th>Wet_3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure load (kN)</td>
<td>1.1</td>
<td>1.7</td>
<td>1.1</td>
<td>0.5</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Average failure load (kN)</td>
<td></td>
<td>1.3</td>
<td></td>
<td>0.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Standard deviation (kN)</td>
<td></td>
<td>0.3</td>
<td></td>
<td>0.1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.4.4 Shear bond properties investigation

It is common knowledge that fractures and joints inevitably exist at underground coal mine excavation surfaces. The penetration of TSL into the fractures and joints helps to bond the cracked rock blocks together (Figure 3.24), which is beneficial to the rock stability. In this scenario, the shear bond of the TSL plays an important role in inhibiting the movement of the rock blocks. Thus, it is of significance to determine the shear bond strength of the TSL.
Unlike the tensile and flexural properties of TSLs, there is no standard test method to determine the shear bond strength of TSL material. As mentioned in chapter two, Saydam et al. (2003) proposed a double-sided shear strength (DSS) test. This test involved gluing three rock blocks with TSL material at adjacent surfaces and pressuring the middle block till de-bonding occurs. The disadvantage of this method was that bending may result in premature failure, making it difficult to interpret the test results. Yilmaz (2011) presented a new testing approach, the principle of which was illustrated in chapter two. This method was able to avoid the bending issue in the test of Saydam et al. (2003), however, it ignored the confining stress produced by the shrinkage of the polymer material. In this section, a modified version of the DSS test method was proposed to evaluate the shear bond strength of the TSL. Furthermore, the effect of moisture on the shear bond strength of the TSL was also investigated.

3.4.4.1 Test fixture and sample assembly

Figure 3.25 shows the test fixture and sample assembly. Two steel support blocks with internal threads were welded onto the steel base plate with the dimensions of 250 mm × 100 mm × 20 mm. The two steel blocks were 70 mm × 95 mm × 45 mm in size and were 50 mm apart. The sample was centrally placed onto the steel support blocks and clamped tightly with the steel clamping plates using 4 screws on each side. The steel clamping plates were 75 mm × 100 mm × 20 mm. A steel loading block, 40 mm × 40
mm × 60 mm, was placed squarely on the central sandstone cube of the sample and the load was applied.

![Double-sided shear strength test fixture](image)

**Figure 3.25** Double-sided shear strength test fixture

In the DSS test of Saydam (2003), there was only one clamp used to tighten the side rock block, which may not have been able to prevent bending from occurring. However, four bolts were employed in this study to tighten the side blocks to eliminate any potential bending issue.

3.4.4.2 Preparation of samples

Sample preparation was an important part of the whole testing program. The validity of the test results largely depends on the preparation of the samples. The samples for double sided shear test were prepared following the procedures below:

(i) Sandstone cubes, 40 mm × 40 mm × 40 mm (Figure 3.26b), were cut from large sandstone blocks (Figure 3.26a). It was important to make sure that each surface was perpendicular to the adjacent surfaces. The cubes were divided into two groups. While one group (dry group) was kept in the room environment, the other group (wet group) was saturated in water for 24 hours.
(ii) The cubes were wrapped with masking tape on the faces where bonding was not desired (Figure 3.26c). As such, the polymer would not get into contact with these faces. Before putting the cubes into the steel mould, wax was applied onto the steel mould to ensure easy sample removal after the polymer cured. After that, three cubes were placed in the steel mould with a 5 mm gap between them.

(iii) The polymer was mixed in accordance with the manufacturer’s instructions. For polymer A, it was transported into the gap by the use of a small spatula. For polymer B, it was poured into the gap directly. The samples were kept in the mould overnight.

(iv) The samples were taken out of the mould and left in the room environment for two weeks to allow the polymer to fully cure.

Figure 3.26 Procedures of sample preparation
3.4.4.3 Test set-up and procedure

Figure 3.27 shows the test set-up. An Instron machine was utilized to conduct the test. The sample was carefully positioned in the test apparatus and tightly clamped by the bolts. The clamp should neither be too tight to break the side sandstone cube nor be too loose to allow bending. Displacement control was applied at a loading rate of 0.005 mm/s, a typical test completed within 5 minutes. Load and displacement were recorded during the test.

![Test set-up image]

Figure 3.27 Doubled-sided shear bond test set-up

3.4.4.4 Calculations

In this test, the surfaces of the sandstone cubes were assumed to be identical in terms of surface condition. Thus, the detachment of the sandstone surface from the polymer surface should theoretically occur simultaneously at both the two sides. The shear bond strength can be calculated by dividing the load at failure by two times the area of the interface, using the following equation:

$$\sigma_{sb} = \frac{P}{2A_i}$$  \hspace{1cm} (3.13)

Where:

$\sigma_{sb} =$ the shear bond strength (MPa)
\( P \) = the load at failure (N)
\( A_i \) = the area of the interface (mm\(^2\))

### 3.4.4.5 Failure mode

Three failure modes were observed during the tests. They were shear bond failure (Figure 3.28a), quasi shear bond failure (Figure 3.28b) and failure in the sandstone (Figure 3.28c). The first failure mode referred to de-bonding occurring at the polymer-sandstone interface and there was neither polymer left on the sandstone surface nor sandstone glued on the polymer. This was the ideal failure mode as the shear bond strength was able to be accurately calculated by dividing the failure load by the total area of the interface. In the second failure mode, the failure did not completely occur at the polymer-sandstone contact area but a very slight (less than 10%) failure was in the sandstone or in the polymer layer. This failure mode was characterized by a little part of sandstone glued on the polymer or a little section of polymer glued on the sandstone after failure. Usually, this failure mode initiated with shear failure in the polymer at first and then followed by the shear bond failure at the sandstone-polymer interface. For the last failure mode, most of the failure (more than 90%) occurred in the sandstone during the testing as the shear bond strength of the polymer was greater than the cohesion of the sandstone. In this failure mode, shear failure in the polymer was discovered at first and then transits to shear failure in the sandstone.

![Failure modes](image)

Figure 3.28 Failure modes of the sample (a) shear bond failure, (b) quasi shear bond failure and (c) failure in the sandstone

### 3.4.4.6 Results and discussion

Load-displacement curves of polymer A bonded to dry sandstone samples are given in Figure 3.29. Both sample two and three experienced quasi shear bond failure while
sample one failed in the sandstone. Sample two and three had similar peak loads, this was because one side of the centre sandstone block failed initially and then followed shortly after by failure at the other side. It is apparent from the figure that the peak loads of sample two and three were much greater than that of sample one. Specifically, while the peak loads of sample two and three were around 32 kN that of sample one was approximately 20 kN. The low peak load of sample one was due to the fact that fractures developed in the two side sandstone blocks prior to testing, as a result of an excessive clamping force applied in the sample assembly process. As shown in the figure the load did not fall to zero directly but decreased slowly after the failure of all three samples. This was attributed to the friction resulting from rough broken surfaces.

![Figure 3.29 Load-displacement curves of polymer A bonded to dry sandstone samples](image)

The load versus displacement curves of polymer A bonded to wet sandstone samples are described in Figure 3.30. As expected, both samples experienced shear bond failure and the failure loads were very low. The peak loads achieved were around 1.9 kN which was only approximately 5.9% of that for the dry samples, indicating that the water content at the interface had a significantly detrimental effect on the shear bond.
strength. The water content at the interface prevented intimate contact between the substrate and the polymer, which had a detrimental effect on the intermolecular bonding between the two materials and eventually weakened the shear bond strength.

Figure 3.30 Load-displacement curves of polymer A bonded to wet sandstone samples

Figure 3.31 illustrates the load-displacement behaviours of polymer B bonded to dry sandstone samples. None of the three samples failed in the first failure mode. Sample one experienced the second failure mode, and samples two and three experienced the third failure mode. Compared with sample one and three, the peak load of sample two was much smaller, which was due to the pre-mature failure in the side blocks, prior to the testing, again resulting from excessive clamping. The peak load of sample one reached approximately 42 kN which was 30% higher than that of dry polymer A samples. This was probably because polymer B was less viscous than polymer A, which made it relatively easier to seep into the sandstone grains and as such created a stronger bond at the interface between the polymer and the sandstone.
Figure 3.31 Load-displacement curves of polymer B bonded to dry sandstone samples

The load-displacement curves of polymer B bonded to wet sandstone samples are presented in Figure 3.32. As expected, all of the three samples experienced the first failure mode. The maximum load achieved in this group of test was much lower than that gained in the dry group, being 3 kN, 4.2 kN and 8.4 kN for sample 1, 2 and 3 respectively. It is worthwhile to note that the failure load of sample three was much greater than both sample one and sample two. This was because while sample one and two were bonded to the polymer immediately after taking out of the water, sample three was placed in the room environment for more than one hour after being removed from the water and before applying the polymer.

Figure 3.32 Load-displacement curves of polymer B bonded to wet sandstone samples
For samples failed in the third failure mode, it was not possible to calculate the shear bond strength of the polymer to the sandstone. For samples failed in the second failure mode, the shear bond strength of the polymer was calculated using Equation (3.13), although not completely accurate the error can be considered negligible. Table 3.11 shows the shear bond strength of the samples. It is obvious that the wet surface of the sandstone significantly decreased the shear bond strength of both polymers. Specifically, while the dry polymer A sample had a shear bond strength of 10.6 MPa, wet polymer A sample experienced a reduction of 94% in shear bond strength being 0.6 MPa. The shear bond strengths of dry polymer B and wet polymer B samples were 13 MPa and 1.6 MPa respectively, indicating a drop of 88% when applied to the wet interface. The relatively lower sensitivity of polymer B than polymer A bonding to wet sandstone surface was attributed to the lower viscosity of polymer B. Due to the higher viscosity, polymer A had smaller shear bond strength than polymer B whether the interface was dry or wet. The shear bond strength of dry polymer A sample was 10.2 MPa which was 22% lower than that of dry polymer B sample. The shear bond strength of wet polymer A sample was also 63% lower than that of wet polymer B sample.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Shear bond strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry polymer A</td>
<td>N.A.</td>
</tr>
<tr>
<td>Wet polymer A</td>
<td>0.6</td>
</tr>
<tr>
<td>Dry polymer B</td>
<td>13.0</td>
</tr>
<tr>
<td>Wet polymer B</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Table 3.11 Shear bond strength of TSLs

Based on the test results, it is clear that moisture has negative influence on the TSL shear bond strength. Thus, it is recommended that the excavation surface be as dry as practicable before applying the TSL.
It is worthwhile to note that the shear bond strength difference between dry and wet samples was much greater than the adhesion strength difference between the two groups of samples. This was probably because that the gravity helped the polymer to saturate into the pore of the wet adhesion test sample, creating bonding between the polymer and the sandstone. However, this mechanism did not exist in the wet samples for shear bond strength test.

3.5 Determining the mechanical properties of the steel wires used in weld mesh

As numerical modelling was conducted in this study to evaluate the behaviour of full scale steel mesh in the pull test, this section was an attempt to determine the input parameters and related information for the numerical model by doing laboratory tests. The laboratory tests conducted included tensile tests on 5 mm and 7 mm diameter steel wire, three-point bending test on 5 mm and 7 mm diameter steel wire and weld shear tests. The steel wires were cut from weld mesh currently used in local underground coal mines.

3.5.1 Tensile test on steel wires

Previous study (Villaescusa 2004) found that welded mesh has three failure modes which include tensile failure of the wire, shear failure at the weld points and failure on the Heat Affected Zone (HAZ). The tensile failure of the wire depends on its tensile strength, and the load capacity of the mesh is a function of the wire tensile strength. In this study, the tensile characteristic of two types of steel wires with different diameters (5 mm and 7 mm) was investigated.

3.5.1.1 Test set-up and procedure

The tensile test was conducted according to Australian Standard AS 1391-2007, Metallic materials-Tensile testing at ambient temperature. The test set-up is shown in Figure 3.33. An extensometer was applied during the testing to record the strain of the wire. The load was applied in displacement control model. The initial loading rate was 2 mm/min up to yield and then increased to 3 mm/min to failure. The increase in the loading rate was to ensure the test completed in a reasonable time period.
3.5.1.2 Test results

The tensile strengths of 5 mm and 7 mm diameter steel wires are shown in Table 3.12. Five samples for each wire diameter were tested, and it was found that the maximum tensile stress reached by the 5 mm diameter steel wire was 460 MPa and that of 7 mm diameter wire was 560 MPa. As expected the Young’s Moduli of the wires were both around 200 GPa.

Table 3.12 Tensile strength of 5 mm and 7 mm steel wires

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>5 mm steel wire</th>
<th>7 mm steel wire</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Maximum tensile stress (MPa)</td>
<td>464</td>
<td>463</td>
</tr>
<tr>
<td>Average (MPa)</td>
<td>460</td>
<td></td>
</tr>
<tr>
<td>Standard deviation (MPa)</td>
<td>6</td>
<td></td>
</tr>
</tbody>
</table>
3.5.2 Bending test on steel wires

3.5.2.1 Test set-up and procedure

In addition to tensile loading, the wire of steel mesh is also subject to bending in situ. Thus, it was necessary to study the behaviour of steel wire under bending. In this section, three-point bending test were conducted on the two types of wire (5 mm and 7 mm in diameter). The bending tests were done in accordance with Australian Standard AS 2505.2-2004, Metallic materials-Method 2: Bars, rods and solid shapes-Bend tests with some modifications. The test set-up is illustrated in Figure 3.35. The steel wire was supported by two rollers with a span of 130 mm, and the load was applied at 10 mm/min, the test terminated at 50 mm displacement. The load and the corresponding displacement curves were recorded during the test.

![Figure 3.34 Bend test set-up](image)

3.5.2.2 Test results and discussion

The load versus displacement curves of testing on 5 mm steel wire and 7 mm steel wire are shown in Figure 3.35 and Figure 3.36 respectively. It was apparent from the figures that the 7 mm diameter steel wire was stronger than 5 mm diameter strand. Specifically, while the yield load of the thinner wire was approximately 0.4 kN the yield load of the thicker wire was around 1.25 kN. Displacements at yield for the two
types of steel wire were similar being around 2.7 mm. From beam theory, the
displacement at yield can be calculated using the following equation:

\[ y = -\frac{P_y l^3}{48EI} \]  \hspace{1cm} (3.14)

Where:

\( y \) = the displacement at yield (mm)
\( P_y \) = the yield load (N)
\( l \) = the support span (mm)
\( E \) = the young’s modulus (MPa)
\( I \) = the second moment of area (mm\(^4\))

Substituting the corresponding values into the equation (note: the diameter for the
thinner steel wire was taken as 5.3 mm, the measured value) the displacements at yield
for the two wires were predicted to be 2.40 mm and 2.43 mm respectively, which were
very close to the laboratory results.

Figure 3.35 Load versus displacement curves of testing on 5 mm steel wire
Figure 3.36 Load versus displacement curves of testing on 7 mm steel wire

3.5.3 Weld shear test

3.5.3.1 Test set-up and procedure

As mentioned above, weld shear failure is one of the three failure modes of weld mesh. The weld shear strength of the mesh also plays an important role in rock support as it affects the load transfer mechanism. The tests were based on the Australian Standard AS 1304-1991 in which it is suggested that ‘the minimum breaking load in Newtons (N) shall not be less than 250 multiplied by the nominal area of the longitudinal wire in square millimetres’. The test apparatus used in this study was a simple version of that suggested in the standard, and it is shown in Figure 3.37 (a). Figure 3.37 (b) illustrates the test set-up.
3.5.3.2 Test results and discussion

Test results are presented in Table 3.13. Eight samples were tested with 7 samples undergoing failure at the Heat Affected Zone (HAZ) and the other one breaking due to tensile failure of the wire. None of the samples experienced weld shear failure, which indicated that the weld shear strength was greater than the failure load achieved in the tests. As such, weld shear failure would not be considered in the numerical modelling. It was also noted from the table that the failure load of the HAZ failure mode was very close to that of the wire failure mode. Thus, it was decided that considering the HAZ failure in the numerical modelling would be of no benefit, but just add an unnecessary layer of complexity.

Table 3.13 Weld shear test results

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure mode</td>
<td>HAZ</td>
<td>HAZ</td>
<td>HAZ</td>
<td>HAZ</td>
<td>HAZ</td>
<td>HAZ</td>
<td>wire</td>
<td>HAZ</td>
</tr>
<tr>
<td>Failure load (kN)</td>
<td>10.0</td>
<td>10.0</td>
<td>10.1</td>
<td>10.0</td>
<td>10.0</td>
<td>10.2</td>
<td>10.2</td>
<td>10.2</td>
</tr>
<tr>
<td>Average failure load (kN)</td>
<td>10.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Standard deviation (kN)</td>
<td>0.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.6 Summary and conclusions

Uniaxial compressive tests, triaxial compressive tests and Brazilian disc tests were conducted on two types of sandstone and hydrostone plaster frequently used in the laboratory. The UCS, tensile strength, cohesion, Young’s modulus and Poisson’s ratio of the materials were determined. The test results provide input parameters for the numerical modelling to be covered in this study and also help to analyse results of other tests in this study.

As mechanical properties of the hydrostone plaster are influenced by the curing time and environment, additional laboratory tests were performed to investigate the change of UCS and flexural properties of hydrostone plaster as a function of curing time and environment, the following conclusions can be drawn:

- Higher temperature makes the hydrostone plaster cure quicker and produces stronger samples in both compression and flexure;
- The plaster increased in UCS and flexural strength over time to a limit, experienced a stable stage and then experienced a degradation of strength after around 6 to 8 weeks;
- When plaster is utilised to simulate rock materials it is important to make sure they are tested in the stable stage.

Three types of TSL prototype material, polymer A, B and C, and their fibre reinforced versions were subjected to tensile, three-point bending, adhesion and shear bond tests. The test results indicated that a significant increase in the tensile strength, Young’s modulus, flexural strength and flexural modulus was achieved when polymer A and B were reinforced with glass fibre sheets. Compared with polymer A and B, polymer C had a relatively small increase in the tensile and flexural strength, and Young’s and flexural moduli when reinforced with bar-chip fibre. This was attributed to the lesser amount of bar-chip fibre (4.3%) that was able to be added to the very viscous polymer. It was believed that polymer C would be stronger if more bar-chip fibre was applied as was the case for polymer B which was stronger when reinforced with 3 layers of fibre sheet than when reinforced with 2 layers of fibre sheet. This was supported by the results of tensile tests on polymer C reinforced with 3 layers of fibre sheet (15.8%)
and produced much greater tensile strength. The increase in the strength of the TSL is beneficial to the rock support in underground coal mines as the TSL is able to bear greater load. The increase in the Young’s and flexural moduli is also desirable as the TSL would have less deformation when subject to the same load, thus preventing the rock mass from dilating and ultimately help to preserve the inherent strength of the strata.

The fibre reinforced polymers exhibited post failure behaviour in the three-point bending test, which made them fundamentally different from the unreinforced polymer. The post failure behaviour of the TSL is desirable for underground coal mine roof support, because it does not fail in a brittle way and experiences large deformation before breaking completely. While it was clearly demonstrated that fibre reinforcement helped to increase the tensile strength and Young’s modulus of the TSLs, its effect on the Poisson’s ratio was not significant. All of the various types of plain polymer and fibre reinforced polymer had the Poisson’s ratios ranging around 0.35 ± 0.03.

Strong adhesion of the TSL is of significant importance in strata control as it ensures the TSL is tightly adhered to the rock surface. It will be shown later that when the TSL is bonded to the rock surface, the two materials form a composite which is stronger than the sum of the parts. The exact adhesion strength of polymer A and B to the sandstone was not able to be evaluated by the laboratory experiments in this study as all of the failure occurred in the sandstone rather than at the polymer-sandstone interface. This indicated that the adhesion strength of both types of polymer were greater than the tensile strength of the sandstone, which was 3.1 MPa. It is believed that failure at the TSL-substrate interface would be achieved if stronger rock was used as the substrate, however, the substrate in a coal mine is generally coal, sandstone or some other relatively weak rock.
As bending influences the results in the DSS test method proposed by Saydam et al. (2003), a modified version of the DSS test was successfully used by modifying the test to increase the clamping force of the outer cubes. The shear bond strength of polymer A and B on dry and wet sandstones were investigated. Polymer B was found to have a stronger shear bond than polymer A on both dry and wet sandstones. This was probably due to the relatively lower viscosity of polymer B making it easier to penetrate into the layer of the substrate and to bond with the grains of the sandstone. The test results also indicated water content had a detrimental influence on shear bond of the polymers. The shear bond strength of polymer A was reduced by 94% and that of polymer B was decreased by 88% when the substrate was saturated sandstones, indicating the excavation surface should be as dry as practical before the TSL is applied, otherwise a priming layer may be required.

Tensile tests on 5 mm and 7 mm diameter steel wires which were used in the manufacture of weld mesh for roof support in Australian underground coal mines were tested in the laboratory. As expected, the Young’s moduli of the two types of steel wires were calculated to be approximately 200 GPa. The tensile strengths of 5 mm and 7 mm diameter wires were 460 MPa and 560 MPa respectively, indicating that they were formed from different types of steel. The three-point bending tests on the two types of wire showed that the yield load of the 7 mm diameter steel wire was much greater than that of the 5 mm diameter wire. The weld of the steel mesh was of very good quality as none of the 8 samples tested experienced weld shear failure. As such, it was not necessary to consider the weld shear failure in the numerical modelling in this study.

The tests results in this chapter are able to either directly provide input parameters or indirectly provide useful information for the numerical modelling in this study. The following chapter investigates the reinforcement capacity of the TSL and steel mesh in rock support.
4 INVESTIGATION INTO THE ROCK SUPPORT CAPACITIES OF THIN SPRAY-ON LINERS AND STEEL MESH

4.1 Introduction

Roof instability has always been a major issue in underground coal mines. The failure modes of the roof in underground coal mines have been widely studied. Zhang and Peng (2002) classified the roof failure into five types: skin failure, buckling failure, cutter roof failure, shear failure and compressive shear failure. According to Shen (2014), there are six types of roof failure mechanisms. They are beam failure which often occurs in bedded rock roofs, joint controlled rock falls, roof sag, guttering and shear failure, and skin failure. Among these failure modes, beam failure of weak bedding planes, buckling and guttering are commonly encountered in underground coal mines. They have detrimental influence on the roadway stability, and may even threaten the safety of personnel, therefore, it is important to investigate the ability of any newly developed roof support medium, such as thin spray-on liners (TSLs), in supporting roof with these geological features.

In this chapter, the beam enhancement mechanism of a TSL is studied using four-point bending test in the laboratory. The effect of the TSL, which forms a composite beam with the roof rock after application, on helping the roof beam resist bending is evaluated. One of the advantages of thin spray-on liners (TSLs) over welded steel mesh in underground roof support is that TSLs can be sprayed into the cracks in the rocks. However, the mechanism by which thin spray-on liners help to stabilise the ‘cracked’ roof or rib of an underground tunnel is not completely understood. To gain a better understanding, a series of plaster beams with different notch shapes and notch reinforcement were tested to failure in a four-point bending test.

Although TSLs are believed to have the potential to take the place of steel mesh in underground roof support, the reinforcement capacity of TSLs in rock support has not been studied adequately. Publications have mainly focused on determining the
mechanical properties (tensile strength, shear strength, adhesion strength and shear bond strength) of various TSLs. Investigation on the behaviour of TSLs in reinforcing typical rock structures in underground coal mines has been limited. Direct comparison between the innovative (TSLs) and traditional (steel mesh) strata control methods has been scarce. As such, a series of laboratory tests were designed and performed to compare the behaviours of TSLs and steel mesh in supporting underground coal mine roof with weak bedding planes, buckling and guttering in this chapter.

4.2 Beam enhancement capability of a thin spray-on liner and its ability to support jointed roof strata

4.2.1 Methodology

In a three-point bending test, the peak stress occurs at the middle of the sample and the stress elsewhere is lower, however, a four-point bending test has a larger peak stress region located in the pure bending zone, making it possible for more defects in the specimen to be affected by the peak stress. As such, four-point bending test was chosen instead of three-point bending test to evaluate the beam enhancement capability of the TSL. Three-point bending test was selected in chapter three was because it was relatively easier to conduct and it was enough to evaluate the bending behaviours of the plaster.

Polymer A reinforced with 3 layers of glass fibre sheet was used to simulate the TSL in this test. Figure 4.1 illustrates the schematic of the four-point bending test. The sample tested in this experiment was rectangular beam with dimensions of 160 mm in length, 40 mm in thickness and 40 mm in width.
Three series of eight groups of sample were tested. For the sake of simplifying the description, an abbreviation of the name of each group was applied in this study and are listed in Table 4.1. The first series was to evaluate the beam enhancement capacity of the TSL. It included two groups of sample, which were plain plaster beams (PP) serving as the control group and fibre reinforced polymer (FRP) A reinforced plaster beams (PT). The second series, which was to investigate the TSL in reinforcing samples with a rectangular notch, consisted of three groups of sample. They were plaster beams with rectangular notch (PR) serving as the control group, and FRP reinforced plaster beams with rectangular notch (PTR) and FRP reinforced plaster beams with polymer filled rectangular notch (PTPoR). The third series was to study the TSL in supporting samples with v-shaped notch. It included three groups of beams, which were plaster beams with v-shaped notch (PV) serving as the control group, FRP reinforced plaster beams with v-shaped notch (PTV) and FRP reinforced plaster beams with polymer filled v-shaped notch (PTPoV).
Table 4.1 Abbreviation of each group sample

<table>
<thead>
<tr>
<th>Series name</th>
<th>Full group name</th>
<th>Abbreviation of group name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam enhancement</td>
<td>Plain plaster beam</td>
<td>PP</td>
</tr>
<tr>
<td></td>
<td>FRP reinforced plaster beam</td>
<td>PT</td>
</tr>
<tr>
<td>Rectangular notch</td>
<td>Plaster beam with rectangular notch</td>
<td>PR</td>
</tr>
<tr>
<td></td>
<td>FRP reinforced plaster beam with rectangular notch</td>
<td>PTR</td>
</tr>
<tr>
<td></td>
<td>FRP reinforced plaster beam with polymer filled rectangular notch</td>
<td>PTRoR</td>
</tr>
<tr>
<td>v-notch</td>
<td>Plaster beam with v-shape notch</td>
<td>PV</td>
</tr>
<tr>
<td></td>
<td>FRP reinforced plaster beam with v-shape notch</td>
<td>PTV</td>
</tr>
<tr>
<td></td>
<td>FRP reinforced plaster beam with polymer filled v-shape notch</td>
<td>PTPoV</td>
</tr>
</tbody>
</table>

4.2.2 Sample preparation

Plaster, which is a polycrystalline material made of intricate gypsum needles, is often used as a rock mechanics modelling material (Coquard & Boistelle 1994) in the laboratory. Its popularity in the laboratory is because it is easy to configure the desired sample shape. In these tests hydrostone plaster, a mix of gypsum plaster and 5% Portland cement, was used.

All the hydrostone plaster beams were prepared in a similar way to the preparation of the cylindrical plaster samples described in section 3.2, except that a different steel mould (Figure 4.2a) was used. In order to facilitate easy removal of the plaster beam from the steel mould after curing, a thin layer of vaseline was applied onto the mould surface before pouring the mixture of plaster and water. The pouring process should be gentle so as to eliminate the formation of air bubbles in the beam. The plaster beam was left to cure in the mould for 30 minutes before it was removed. As indicated in chapter three the flexural strength of the plaster beam is relatively stable after four
weeks under room cure conditions, thus all the plaster beams were stored in the room environment for four weeks before being subjected to the bend test. The samples of each group were prepared as follows:

(i) For the PP, which acted as the control group, no additional preparation was needed after being removed from the steel mould, except for the four-week curing in the room environment.

(ii) With respect to the PV and PR, two notches representing different fractures were machined into the beams. One notch was rectangular with a width of 1 mm and a depth of 10 mm, the other one was a ‘v’ shape and its angle and depth were 10° and 10 mm respectively. Both notches were machined in the middle of the bottom of the beams.

(iii) When it came to PT, a more complicated procedure was involved in the sample preparation. The plaster beam was trimmed by 5 mm in thickness after it was removed from the steel mould. The dimensions of the beam were now 160 mm in length, 40 mm in width and 35 mm in thickness. After that, the beam was placed into the mould again and a 5 mm thick layer of polymer A reinforced with 3 layers of glass fibre sheet was bonded to the bottom face. This process was identical to the preparation of the FRP described in chapter three. A steel plate was placed onto the top surface of the polymer skin to make sure the surface was smooth (Figure 4.2b). Note that vaseline was also applied on the bottom surface of the steel plate to ensure easy separation after the polymer cured. The sample was cured in the mould for 24 hours before removing (Figure 4.2c). Four weeks were allowed for the sample to cure prior to testing.

(iv) The preparation of PTV and PTR was slightly different from that of PT. The difference lay in that a rectangular or ‘v-shape’ notch was machined into the plaster beam at the centre of its bottom surface after it was cut to 35 mm thick. The notch was filled with ‘blu-tack’ prior to the bonding of the FRP, which was to avoid the polymer filling the notch. The filling material was removed from the sample after 24 hours of curing. Figure 4.2d shows the PTV sample with dimensions illustrated.
(v) PTPoV and PTPoR were prepared in a similar way with those of PTV and PTR except that polymer was allowed to fill the notch to mimic the penetration of the TSLs into the fractures of the rock mass.

Figure 4.2 Preparation of the sample

4.2.3 Test set-up and procedure

The four-point bending test set-up is shown in Figure 4.3. An Instron machine was employed to conduct the tests. In this test, the prepared sample was symmetrically supported by two steel rollers with a span of 120 mm at the bottom surface; the rollers were fixed to a solid base. Two other steel loading rollers were placed centrally on the top surface of the sample, the span between the two rollers was 40 mm. Displacement control was selected, and the load was applied to the sample by pushing a hydraulic
actuator downwards. The load and the displacement of the loading roller were recorded during the test. The loading rate for the unreinforced plaster beam was 0.1 mm/min, and the rate for the reinforced samples was initially 0.1 mm/min to sample yield and then increased to 0.4 mm/min up to failure.

Figure 4.3 Four-point bending test set-up

4.2.4 Results and discussion

4.2.4.1 Beam enhancement capacity of thin spray-on liner

The typical load versus displacement curves of the four-point bending test on plain plaster beams (PP) and FRP reinforced plaster beams (PT) are shown in Figure 4.4. It was apparent that PT was stronger in terms of peak load and showed more deformability, compared with PP. Moreover, PT exhibited a post-failure behaviour while PP failed in a brittle way. The load-displacement curve of FRP reinforced beams can be divided into three stages, as shown in the figure. Stage one was a quasi-elastic stage, at this stage, there was no failure in neither the plaster nor the FRP and the displacement increased quasi-linearly with the applied load. Stage two was the crack propagation stage. It started from the first crack in the reinforced sample, and it began with the first load drop in the curve. At this stage, small cracks initially developed in the plaster beam and then propagated as the load increased. As shown in the figure, there were many saw-teeth in the curve, which corresponded to the development of new cracks or existing crack propagation. The FRP may also crack at the support roller due to stress concentration in this stage. Stage three was the ultimate stage. This stage started with the structural failure of the sample which was indicated by the rapid drop
in load. It is important to note that due to the FRP reinforcement the FRP-plaster composite beam had a significant residual strength, which is desirable for underground coal mine roof support (Figure 4.4).

![Graph showing load-displacement characteristics of plain and reinforced plaster beams](image)

Figure 4.4 Typical load-displacement behaviours of plain and reinforced plaster beam

The detailed load versus displacement curves of PP and PT in the four-point bending test are shown in Figure 4.5 and Figure 4.6 respectively. The failure load and maximum load of the samples and their corresponding displacements are listed in Table 4.2. It is obvious from the figures and table that the test results were similar, specifically, the failure loads of the PP samples were 4.87 kN, 4.96 kN and 4.91 kN, with a standard deviation of 0.05 kN. The displacements at failure of the PP samples were also similar, being 0.48 mm, 0.48 mm and 0.52 mm respectively with a standard deviation of 0.02 mm. The PT samples were greater than PP samples in terms of standard deviations for both the maximum load and the corresponding displacement, however, the values were still small considering the average maximum load of 11.18 kN and an average displacement at maximum load of 2.13 mm. The average peak load of the FRP reinforced beam was 2.3 times the average failure load of the plain control beams. This clearly demonstrated the significant beam enhancement capacity of the TSL. The TSL-plaster composite beams had also shown significant residual strength
ranging from about 1.85 kN to 4.51 kN. This is beneficial to roof support as the post failure behaviour is able to help eliminate the abrupt fall of the failed roof rock, so as to improve mine safety. The variation in the residual strength among the three samples could be due to the differences in the strength of the FRP, the plaster beam and the bond strength at the interface of the two components of the composite.

Figure 4.5 Load-displacement curves of PP

Figure 4.6 Load-displacement curves of PT
Table 4.2 Results of four-point bending test on PP and PT

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Average</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>PP</td>
<td>Load at failure (kN)</td>
<td>4.87</td>
<td>4.96</td>
<td>4.91</td>
<td>4.91</td>
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<tr>
<td></td>
<td>Displacement at failure (mm)</td>
<td>0.48</td>
<td>0.48</td>
<td>0.52</td>
<td>0.49</td>
</tr>
<tr>
<td>PT</td>
<td>Peak Load (kN)</td>
<td>11.65</td>
<td>11.26</td>
<td>10.62</td>
<td>11.18</td>
</tr>
<tr>
<td></td>
<td>Displacement at peak load (mm)</td>
<td>2.23</td>
<td>1.96</td>
<td>2.21</td>
<td>2.13</td>
</tr>
</tbody>
</table>

The normal stress distribution at the cross section of PP is illustrated in Figure 4.7(a). The maximum tensile and compressive bending stresses occurred at points located farthest from the neutral axis. In this study, the maximum tensile stress acted at the bottom surface of the plaster beam, the maximum compressive stress acted at the top surface of the beam. As the compressive strength of the plaster was greater than its tensile strength, the maximum tensile stress which occurred in the beam was its flexural strength. The flexural strength of PP can be calculated from the following equation:

\[
\sigma_f = \frac{3P_f l}{4bt^2}
\]

(4.1)

Where:

- \(\sigma_f\) = flexural strength (MPa)
- \(P_f\) = failure load (N)
- \(l\) = span of the roller support (mm)
- \(b\) = width of the plaster beam (mm)
- \(t\) = thickness of the plaster beam (mm)
The support span was 120 mm, and both the width and thickness of the plaster beam were 40 mm. Substituting the failure loads of sample one to sample three gives the flexural strength of sample one as 7.2 MPa, sample two as 7.1 MPa and sample three as 7.0 MPa. As mentioned above, the flexural strength was the maximum tensile stress at the bottom surface of the rectangular plaster beam in the four-point bending test. The tensile stress calculated here was slightly greater than that in Table 3.5. The reason may have been because the samples in the bending test were cured for a longer time than that in the Brazilian disc test, and it was shown in section 3.3 that plaster was stronger with a longer curing time.

When the plaster beam was bonded to the FRP at the bottom surface, the normal stress distribution at the cross section of the composite material altered and is shown in Figure 4.7(b). As the elastic moduli of the two components of the composite were not identical, the neutral axis of the PT sample was not at the centre of its cross section as was the case for the PP beam. In order to determine the maximum normal stress occurring at the cross section of the PT samples, it was necessary to determine the location of the neutral axis of the beam.
In the four-point bending test, the resultant axial force acting on the cross section is zero; therefore,

$$\int_1 \sigma_1 dA + \int_2 \sigma_2 dA = 0 \quad (4.2)$$

\(\sigma_1\) is the normal stress (bending stress) at the cross section of the plaster and \(\sigma_2\) is the normal stress at the cross section of the FRP.

Since,

$$\sigma_1 = E_1 \epsilon_1, \quad \sigma_2 = E_2 \epsilon_2 \quad (4.3)$$

$$\epsilon = -ky \quad (4.4)$$

Where \(E_1\) is the elastic modulus of the plaster and \(E_2\) is the elastic modulus of the FRP, \(\epsilon\) is the longitudinal strain in the composite beam, \(k\) is the curvature of the composite beam and \(y\) is the distance to the neutral axis. Substituting Equations (4.3) and (4.4) into Equation (4.2) produces,

$$-E_1 \int_1 ky dA - E_2 \int_2 ky dA = 0 \quad (4.5)$$

As the curvature is a constant at any given cross section, it can be cancelled from the equation; therefore,

$$E_1 \int_1 ydA + E_2 \int_2 ydA = 0 \quad (4.6)$$

As mentioned in chapter three, the Young’s moduli of the plaster and the FRP were 29 GPa and 9.4 GPa respectively, by substituting these values into Equation (4.6) \(h_1\) was found to be 18.4 mm. The normal stresses in the composite beam can be evaluated by,

$$\sigma_1 = \frac{MyE_1}{E_1 I_1 + E_2 I_2}, \quad \sigma_2 = \frac{MyE_2}{E_1 I_1 + E_2 I_2} \quad (4.7)$$

Where \(M\) is the moment, and \(I_1\) and \(I_2\) are the second moments of area of the plaster and FRP respectively. Substituting the loads at first crack of the PT samples (being 4.18 kN, 5.06 kN and 4.37 kN respectively), the Young’s moduli of the plaster and FRP and the particular \(y\) value into Equation (4.7), the maximum tensile stresses in the plaster occurring at the bottom surface were 8.5 MPa, 10.2 MPa and 8.9 MPa for
sample one, two and three respectively. Obviously, due to the reinforcement of the FRP, the maximum tensile stresses achieved in the plaster of the composite beam were higher than that in the plain plaster beam.

The unreinforced plaster beams failed with fracture initiating at the bottom once the cracking load was reached. The crack occurred in the pure bending region (Figure 4.8a) due to tensile rupture. When the beam was coated with FRP the crack initiated in the shear span at around the neutral axis as a result of shear failure. With increasing loading, the crack progressively propagated toward the load and support points and formed a major diagonal crack. Eventually, the crack further propagated horizontally from the support point to the side of the beam resulting in the shear off of the plaster block as shown in Figure 4.8(b).

![Figure 4.8 Typical failure modes of (a) plain plaster beam and (b) FRP reinforced plaster beam](image)

4.2.4.2 Thin spray-on liner in reinforcing plaster beams with rectangular notch

Cracks are usually ubiquitous in the rock mass at the excavation surface in underground coal mines. To investigate the effect of a TSL in resisting crack propagation from the tip, a second series of beams which were coated on the notched face with a thin FRP A liner were also subjected to the bending test. The notch was 1 mm in width and 10 mm in depth. Three groups of samples were tested, which were PR, PTR and PTPoR. The PR was served as the control group; the PTR was to simulate
a crack which developed after the application of the TSL, and the PTPoR was to simulate the penetration of TSL into the crack in the rock.

The load versus displacement curves of the tests are shown through Figure 4.9 to Figure 4.11. It is clear from the figure that PR failed in a brittle way. As expected, the crack initiated as a result of stress concentration at the tip of the notch in the PR (Figure 4.12). In contrast to PR, both PTR and PTPoR experienced three stages as described in section 4.2.4.1. Note that there was a slight difference in stage three for the two PTR samples. While sample one exhibited a residual strength of about 0.93 kN, sample two did not show residual strength. This was due to the bond between the FRP and the plaster not being strong enough and the left side of the block sheared off (as shown in Figure 4.13) at the end of test two. It is obvious from Figure 4.13 that de-bonding also occurred at the FRP-plaster interface for sample one, however, as the de-bonded area was not big enough for the broken plaster block to shear off, the sample was still able to resist load after peak load was achieved. The three PTPoR samples also experienced three different failure procedures in stage three, correspondingly their failure modes were slightly different (Figure 4.14). For the first sample, de-bonding took place at the FRP-plaster interface and the de-bonded area was approximately 40 mm by 40 mm, the left part of the plaster sheared off soon after the peak load was reached. As such, there was no residual strength recorded. Sample two and sample three had different post failure behaviours in stage three. While sample two experienced a sharp decrease in load at first and then kept stable at a level of 1.26 kN, the load gradually reduced for sample three. This was attributed to the fact that sample two had a de-bonded area of about 40 mm by 20 mm but there was no de-bonding occurring in sample three. Thus, it can be concluded that shear bond strength affected the post failure behaviour of the FRP-plaster composite beam. In underground coal mine roof support, if the shear bond between the TSL and the broken rock block at the excavation surface was lost the lateral restraint to the rock would be reduced. Thus, it would be relatively easy for the loose rock block to move or rotate, which has a detrimental effect on the ‘promotion of block interlock’ mechanism proposed by Stacey (2001), and eventually threaten the roadway stability.
Figure 4.9 Load-displacement curves of plaster beam with rectangular notch

Figure 4.10 Load-displacement curves of FRP reinforced plaster beam with rectangular notch

Figure 4.11 Load-displacement curves of FRP reinforced plaster beam with polymer filled rectangular notch
It is apparent from Figure 4.12 to Figure 4.14 that while the notched plaster beams without reinforcement broke at the tip of the notch the reinforced notched samples failed at a place other than the notch tip, indicating that the application of the TSL redistributed the stress and reduced the stress concentration at the notch tip.

Figure 4.12 Typical failure of plaster beam with rectangular notch

Figure 4.13 Failure modes of FRP reinforced plaster beam with rectangular notch
The detailed results of the tests are listed in Table 4.3. As expected, due to stress concentration, the plaster beams with rectangular notch failed at a very low load, averaging at 0.74 kN, which was approximately one fifth of the strength of the plain plaster beam. The notched beam became much stronger after bonding with the FRP and the peak load achieved was around 10.41 kN. Due to the filling of the polymer material into the notch, stress concentration at the notch tip was reduced and the peak load of the PTPoR samples reached up to 11.50 kN. This indicates that the penetration of a TSL into rock joints is able to inhibit the failure of the rock mass. In underground coal mines, a TSL can penetrate into the joints of rock mass, reducing stress concentration at joint tips and helping to distribute the load more evenly.

The displacements at failure for all three PR samples were very close, being 0.10 mm, 0.10 mm and 0.09 mm respectively. The displacement at peak load of the PTPoR samples.
samples was lower than that of the PTR samples, being 1.56 mm and 1.95 mm respectively. The PTPoR samples were generally stiffer than the PTR samples. At a displacement of 1 mm, the resistance generated by PTR and PTPoR was 5.5 kN and 7 kN respectively. As the displacement increased to 1.4 mm, the corresponding resistance grew to 7.1 kN and 9.7 kN. The results indicate that the penetration of TSL material into rock joints increases the stiffness of the TSL-rock system, which suggests greater resistance would be generated at the same rock displacement in this case. This is desirable for underground coal mine roof support as it makes rock displacing or unravelling more difficult. Therefore the integrity of the rock mass and the inherent strength of the rock mass can be maintained.

Table 4.3 Results of four-point bending test on PR, PTR and PTPoR

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Average</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>PR Load at failure (kN)</td>
<td>0.77</td>
<td>0.70</td>
<td>0.75</td>
<td>0.74</td>
<td>0.04</td>
</tr>
<tr>
<td>PR Displacement at failure (mm)</td>
<td>0.10</td>
<td>0.10</td>
<td>0.09</td>
<td>0.10</td>
<td>0</td>
</tr>
<tr>
<td>PTR Peak load (kN)</td>
<td>10.76</td>
<td>10.05</td>
<td>10.41</td>
<td>10.41</td>
<td>0.50</td>
</tr>
<tr>
<td>PTR Displacement at peak load (mm)</td>
<td>1.86</td>
<td>2.04</td>
<td>1.95</td>
<td>1.95</td>
<td>0.13</td>
</tr>
<tr>
<td>PTPoR Peak load (kN)</td>
<td>10.77</td>
<td>11.99</td>
<td>11.73</td>
<td>11.50</td>
<td>0.64</td>
</tr>
<tr>
<td>PTPoR Displacement at peak load (mm)</td>
<td>1.41</td>
<td>1.72</td>
<td>1.56</td>
<td>1.56</td>
<td>0.15</td>
</tr>
</tbody>
</table>

4.2.4.3 Thin spray-on liner in reinforcing plaster beams with a v-shape notch
Another series of tests were conducted on beams with a v-shape notch. Three groups of samples, which included PV, PTV and PTPoV, were tested. The load versus displacement curves of the tests are shown in Figure 4.15 to Figure 4.17. It is obvious from the figures that while PV failed in a brittle way both PTV and PTPoV experienced a three-stage failure process as detailed in section 4.2.4.1. The saw-tooth in the curve indicated crack development or crack propagation. Likewise, as de-
bonding occurred at the interface of some of the FRP-plaster composite beam, some of the FRP reinforced sample did not indicate a residual strength.

Figure 4.15 Load-displacement curves of plaster beam with v-shape notch

Figure 4.16 Load-displacement curves of FRP reinforced plaster beam with v-shape notch

Figure 4.17 Load-displacement curves of FRP reinforced plaster beam with polymer filled v-shape notch
The details of the test results are listed in Table 4.4. PV samples failed at a load of 0.79 kN. As expected, due to the reinforcement of the polymer material, the average peak load of PTPoV was greater than that of the PTV, being 10.74 kN and 10.17 kN respectively. The average displacement at peak load of PTV and PTPoV were 2.18 mm and 1.99 mm respectively.

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Average</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load at failure (kN)</td>
<td>0.84</td>
<td>0.71</td>
<td>0.83</td>
<td>0.79</td>
<td>0.07</td>
</tr>
<tr>
<td>Displacement at failure (mm)</td>
<td>0.09</td>
<td>0.06</td>
<td>0.10</td>
<td>0.08</td>
<td>0.02</td>
</tr>
<tr>
<td>Peak load (kN)</td>
<td>10.50</td>
<td>9.65</td>
<td>10.37</td>
<td>10.17</td>
<td>0.46</td>
</tr>
<tr>
<td>Displacement at peak load (mm)</td>
<td>2.35</td>
<td>2.63</td>
<td>1.56</td>
<td>2.18</td>
<td>0.54</td>
</tr>
<tr>
<td>Peak load (kN)</td>
<td>10.00</td>
<td>11.39</td>
<td>10.83</td>
<td>10.74</td>
<td>0.70</td>
</tr>
<tr>
<td>Displacement at peak load (mm)</td>
<td>2.07</td>
<td>2.26</td>
<td>1.65</td>
<td>1.99</td>
<td>0.31</td>
</tr>
</tbody>
</table>

Figure 4.18 illustrates typical failure of PV, PTV and PTPoV. Like the PR sample, the crack initiated as a result of stress concentration at the tip of the notch for the PV beams. Due to the reinforcement of the FRP, stress was redistributed and there was no crack propagating from the notch in either the PTV or the PTPoV. Inclined shear cracks dominated in both the two composite beams.
Figure 4.18 Typical failure of (a) plaster beam with v-shape notch, (b) FRP reinforced plaster beam with v-shape notch and (c) FRP reinforced plaster beam with polymer filled v-shape notch

4.2.4.4 Discussion

Table 4.5 summarises the average peak loads and corresponding average displacements at peak load for all of the eight groups of samples. It is apparent from the table that the polymer based TSL reinforcement significantly increased the strength of the beams in all cases. The tests indicate that a polymer skin bonded to the plaster surface more than doubles the sample strength while the polymer coated notched samples were more than ten times stronger than the notched plaster sample without the polymer coating. An increase in the strength of the polymer coated sample was observed for notched samples with and without the polymer notch infill. This illustrates the benefits of TSL propagation into the cracks and joints, however, even if the polymer is unable to propagate into the cracks and joints, the TSL still provides a substantial level of reinforcement to fractured strata skin by forming a TSL-rock composite and generating resistance immediately at small rock mass displacement.

Results from the tests also illustrate that while the notch has a negative influence on the deformability of the plaster beam, the application of a TSL is capable of
significantly improving the sample’s load bearing capacity. Overall comparison of the polymer reinforcing capabilities based on the strength of notched or un-notched beams indicates that most of the reinforced samples behaved in a similar manner indicating strong reinforcing capabilities of the polymeric TSL bonded as a composite layer to the beam.

<table>
<thead>
<tr>
<th></th>
<th>Peak load (kN)</th>
<th>Standard deviation (kN)</th>
<th>Displacement at peak load (mm)</th>
<th>Standard deviation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PP</td>
<td>4.91</td>
<td>0.05</td>
<td>0.49</td>
<td>0.02</td>
</tr>
<tr>
<td>PT</td>
<td>11.18</td>
<td>0.52</td>
<td>2.13</td>
<td>0.15</td>
</tr>
<tr>
<td>PR</td>
<td>0.74</td>
<td>0.04</td>
<td>0.10</td>
<td>0</td>
</tr>
<tr>
<td>PTR</td>
<td>10.41</td>
<td>0.50</td>
<td>1.95</td>
<td>0.13</td>
</tr>
<tr>
<td>PTRoR</td>
<td>11.50</td>
<td>0.64</td>
<td>1.56</td>
<td>0.15</td>
</tr>
<tr>
<td>PV</td>
<td>0.79</td>
<td>0.07</td>
<td>0.08</td>
<td>0.02</td>
</tr>
<tr>
<td>PTV</td>
<td>10.17</td>
<td>0.46</td>
<td>2.18</td>
<td>0.55</td>
</tr>
<tr>
<td>PTRoV</td>
<td>10.74</td>
<td>0.70</td>
<td>1.99</td>
<td>0.31</td>
</tr>
</tbody>
</table>

### 4.3 Thin spray-on liner and steel mesh in supporting strata with weak bedding planes

The overburden strata of coal seams are sedimentary rocks. Bedding planes refer to the interfaces between two adjacent rock layers. As the cohesion and friction at the interface are generally very low, it is relatively easy for the bedded roof strata of a roadway to separate at, or slide on, the bedding plane. In situ observations indicate that the effect of bedding planes on the roof stability is at least the same significance or even more than that of the rock type itself (Peng 1998). Brady and Brown (1993) stated that bedding planes, which stand for interruptions in the deposition process, are generally highly persistent structural features of a rock mass. TSLs have been widely studied, however, studies on the performance of TSLs in supporting rock with weak
bedding planes are limited. Therefore, the response of the TSL subjected to this specific structural feature was investigated in this study. The same test was also repeated on a welded steel mesh confined sample to evaluate the potential of a TSL replacing steel mesh.

4.3.1 Experimental programme

Three large scale laboratory tests were conducted to compare the behaviour of a TSL (Polymer A reinforced with three layers of glass fibre sheet) and welded steel mesh in restricting failure due to buckling of strata with weak bedding planes. The three experiments corresponded to three samples; the control sample with no skin confinement, a sample with welded steel mesh reinforcement and a TSL reinforced sample. Figure 4.19 is a schematic of the test. A rectangular concrete block was cast to simulate the rock strata. Several plastic plates were embedded in the block to produce weak bedding planes. A steel frame was mounted at the opposite surface to which the skin confinement was applied, which was to make sure the concrete block buckle toward the confined surface during the loading process. Similar to the in situ situation, the skin confinements were bolted to the surface with rock bolts and load bearing plates. Note that, the load bearing plates and bolt spacing were not identical to those employed in underground coal mines due to the sample size.
4.3.2 Sample preparation

As mentioned above, three samples were prepared for the laboratory testing, which were the control sample, steel mesh confined sample and TSL confined sample. The control sample was a 400 mm × 400 mm × 800 mm rectangular concrete block with embedded thin plastic sheets simulating the weak bedding planes. It was prepared by pouring a mixture of sand, cement and water into a steel mould (Figure 4.20a) while placing the thin plastic sheets at pre-determined positions (Figure 4.20b). After 3 to 4 days of set, four holes were drilled through the concrete block so that the concrete block could have a steel plate attached to the back surface by the use of rock bolts and bearing plates applied to the front surface. The rock bolts were fixed on both sides of the sample. The bearing plates were 110 mm by 110 mm. The other two samples were prepared in a similar way except that welded steel mesh was bolted to one sample and the other sample had a 5 mm thick fibre reinforced polymer (FRP) sheet bonded to the front surface.

For the preparation of the steel mesh confined sample, weld mesh of 780 mm in length and 400 mm in width was firstly cut from a steel mesh sheet which was procured from
a local coal mine. The reason why the mesh was a little shorter than the height of the concrete block was to guarantee that the mesh did not touch the top or bottom platen during loading, so that it was not loaded axially. As with the control sample, four bearing plates and a steel plate were attached to the front and back surfaces of the sample respectively.

In order to bond the 5 mm thick FRP A sheet to the concrete surface, the cast concrete block with the holes filled was placed back into the steel mould. A couple of marks which were just 5 mm higher than the top surface of the concrete block were made on the inner walls of the mould. Prior to the bonding of the polymer, three glass fibre sheets of 780 mm in length and 400 mm in width were cut into size. The reason why the length of 780 mm rather than 800 mm length was used was the same as mentioned in the process of preparation of steel mesh confined sample. Additionally, vaseline was also applied to the inner walls of the mould before the bonding to ensure easy removal of the steel plates of the mould after the curing of the polymer. The polymer material was divided into four equal batches. After the first group of polymer was poured onto the concrete and distributed evenly by the use of a spatula, one glass fibre sheet was placed onto the polymer and gently rolled till the fibre sheet was saturated in the polymer. Then another batch of polymer was applied followed by the application of another fibre sheet. The procedure was repeated so that the three sheets of fibre were embedded in polymer. The steel mould was removed after 24 hours of curing, and then the sample was left to cure for at least one week before drilling holes through the polymer sheet. Finally, four bearing plates and the steel plate were bolted at the front and back surfaces of the sample, as before. All the samples were cured for at least four weeks before testing.
4.3.3 Test procedure

A 500 t Avery compressive testing machine was employed to conduct the test. The sample was placed on a spherical seat which was to make sure that the load is uniformly distributed on the sample during the testing process. Displacement control mode was used in the tests and the loading rate was chosen as 0.5 mm/min, which is slow enough to simulate static loading. The load applied to the sample and the displacement of the platen was monitored using a 5000 kN load cell and a LVDT respectively. The deformation of the centre point of the sample front surface was also recorded by a laser. The test set-up is shown in Figure 4.21.
4.3.4 Results and discussion

The laboratory test results are presented in Table 4.6. The first test was conducted on the control sample. As the load was applied the sample firstly cracked and then small concrete blocks were dislodged. It is worthwhile to note that most of the cracks occurred within the region of the ‘weak bedding planes’ (Figure 4.22a). The peak load achieved in this test was 2494 kN (250 tonnes) and deformation at the centre point of the front surface of the sample (lateral deformation) at peak load was 0.4 mm, the corresponding vertical displacement was 3.1 mm. The test terminated when a structural failure occurred in the sample and the load dropped off dramatically.

During the test on the steel mesh reinforced sample, in addition to small blocks being dislodged in the region of the weak bedding planes, large concrete blocks at the back area were also found to be dislodged (Figure 4.22b). The maximum load recorded in this test was 2321 kN, which was less than that in the first test. Checking the sample after the test showed that one of the rock bolts had sheared at the back, and this was believed to partly attribute to the low strength of the sample. The lateral deformation and vertical displacement at peak load were 1.9 mm and 3.1 mm respectively. Again, the test terminated after structural failure occurred. It is interesting to note that the steel mesh did not undergo much lateral deformation even though the concrete block was seriously broken.

<table>
<thead>
<tr>
<th>Table 4.6 Laboratory test results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<tr>
<td>Control sample</td>
</tr>
<tr>
<td>Peak load (kN)</td>
</tr>
<tr>
<td>Vertical displacement at peak load (mm)</td>
</tr>
<tr>
<td>Lateral deformation at peak load (mm)</td>
</tr>
</tbody>
</table>

In the third test, a 5 mm thick fibre reinforced polymer sheet was bonded to the front surface. The loading process can be divided into three stages. In the first stage, the
polymer sheet adhered well to the concrete and the two materials buckled together in response to the compression. Since the deformation properties of the polymer sheet and the concrete were different, they experienced different deformation under the same loading. When the load reached a certain point the adhesion at the interface of the two materials was not strong enough and de-bonding of the polymer occurred. This was the beginning of the second loading stage. As mentioned above the polymer sheet did not fully cover the front surface of the concrete, a gap existed between the top ends of the two mediums as well as the bottom ends. In this stage the load was carried by the concrete alone. When the concrete deformed to such a degree that it again touched the polymer sheet, the third stage started. During this stage, the two materials together again resisted the loading. As expected, the peak load in this test was greater than that of test one reaching 2856 kN, the corresponding lateral deformation and vertical displacement was 2.0 mm and 3.1 mm respectively. The test halted when the polymer sheet broke along the bottom edge of the steel bearing plate due to shear (Figure 4.22c), that was the only failure in the polymer sheet. Note that the polymer sheet was not substantially deformed after being removed from the concrete, which indicated the sheet was mostly in the elastic region during loading.

Figure 4.22 Failure of the samples
It is apparent from Table 4.6 that the TSL reinforced sample had the greatest peak load, being 362 kN and 535 kN more than the control sample and the steel mesh reinforced sample respectively. The steel mesh reinforced sample experienced the lowest peak load due to the fact that one of the four bolts broke during the loading process. It was therefore not a valid assessment of peak load. Compared with the two other samples the TSL sample was able to accept greater lateral deformation after failure due to the TSL reinforcement.

Figure 4.23 illustrates the relationships of the load versus lateral deformation of the three samples, and Figure 4.24 illustrates the three graphs of load versus vertical displacement. Figure 4.24 shows that the TSL reinforced sample was the stiffest in the load versus vertical displacement relationship, this is because the TSL was bonded to the concrete and it acted as a composite with the concrete at the very beginning of the loading. As the steel mesh did not resist lateral deformation until a certain deformation of the block occurred, the steel mesh reinforced sample should theoretically have similar initial stiffness to that of the control sample. All three samples exhibited post-failure behaviour during the test. While the post-failure strength of the control sample developed due to the rock bolts alone, the strength of the TSL reinforced sample was attributed to the bolts and the polymer sheet while the mesh reinforcement was a function of the welded steel mesh and the bolts. After failure, the control sample could initially still hold a load of around 1200 kN and then gradually decreased to approximately 700 kN. The load bearing capacity after failure of the steel mesh reinforced sample declined from about 1700 kN to 700 kN, while the TSL reinforced sample was relatively stable dropping from about 1500 kN to around 1300 kN.
The test results indicate that a 5 mm thick fibre reinforced polymer sheet provides superior reinforcement than steel mesh to a concrete block with weak bedding planes, suggesting that it would have better performance over welded steel mesh in restricting the softening of fractured strata. As expected, the TSL reinforced block had greater peak load than the control sample with no surface support, however, the peak load experienced by the steel mesh reinforced sample was lower than that of the control.
sample, due to failure of one of the four rock bolts used to bolt the concrete block to
the steel frame. This is a direct indication of rock bolts being the major contributor to
resisting failure of bedded strata. A direct comparison between the TSL and steel mesh
in assisting the bolts to reinforce a fractured rock mass deep within the strata cannot
be made. Failure of the TSL sheet only occurred along the edge of the bolt bearing
plates. It is believed that a plate with more rounded edges would improve the
performance of the TSL.

The vertical loads generated in this test were very large with a maximum load peaking
at nearly 300 tonnes. This resulted in a significant safety risk as the resultant normal
loads caused the rock bolts to shear dynamically. As a result a modified version of this
test was developed, with pre-formed buckling of the strata.

4.4 Thin spray-on liner and steel mesh in supporting buckling strata
Buckling failure occurs in both underground roadway roof and rib primarily due to
high horizontal stresses and overburden abutment pressure respectively. Zhang and
Peng (2002) stated that thinly laminated immediate roof (Figure 4.25) is susceptible
to buckling failure under high horizontal compression. Taking the laminations in the
immediate roof as beams with fixed ends (bolting), the buckling failure criterion can
be derived as:

\[ \sigma_h = \frac{\pi^2 EJ}{bh(a/2)^2} \]  

(4.8)

Where:

\( \sigma_h \) = the horizontal stress in the immediate roof (MPa),

\( E \) = the Young’s modulus of the immediate roof (MPa),

\( J \) = the moment of inertia of the laminated beam (m^4),

\( a \) = the beam length (m),

\( b \) = the beam width (m),
\[ h = \text{the thickness of the beam (m)}. \]

Figure 4.25 Laminated immediate roof

Colwell and Mark (2005) found that lateral displacement of the rib results from rib buckling and the magnitude of the displacement is primarily dependent on the vertical pressure applied to the rib. Buckling failure of the rib has a detrimental influence on roof stability as it increases the roof span. This section is an attempt to compare the relative performance of thin spray-on liners (TSLs) and welded steel mesh in supporting strata subject to buckling.

4.4.1 Experimental programme

A total of 9 samples were tested, two control samples, two steel mesh confined samples, two fibre reinforced polymer (FRP) B confined samples and three FRP C confined samples. The FRP was used to simulate the TSL. Figure 4.26 shows a schematic of the TSL reinforced sample. The sample consisted of four identical plaster slabs with two rubber mats glued to the rear surface and the TSL bonded on the front surface. All the three components were bolted together using bolts and plates. Each plaster slab was 600 mm in length, 200 mm in width and 25 mm in thickness with a 5° slope (Figure 4.27) at the middle to initiate buckling. The rubber mat was applied to prevent loss of plaster fragments during the test.
One of the advantages of TSLs over welded steel mesh in rock support in underground coal mines is that TSLs are bonded to the rock surface, which enables the TSL to respond to even very slight rock displacement, whereas, steel mesh does not generate resistance until a certain magnitude of rock displacement occurs. In order to study this mechanism, the steel mesh was closely bolted to the plaster slab in one sample and an approximately 1 mm gap was left between the mesh and the plaster slab in the other sample. In addition, the plaster slabs of one of the FRP C confined samples were cured for a shorter time to evaluate the influence of rock strength on the behaviour of the TSL.
4.4.2 Sample preparation

Plaster was selected as the substrate material as it is easy to mould into the shape desired. Each plaster slab was made from hydrostone plaster powder and water with the ratio of 3.5:1 by weight. The process of casting each slab was as follows: plaster powder and water with the pre-determined ratio were mixed till homogeneous and then the mixture was gently poured into a mould (Figure 4.28a). The pouring process (Figure 4.28b) should be slow enough to eliminate the entrapment of air bubbles which may form voids in the plaster slab during curing and ultimately weaken the sample. Additionally, grease was applied onto the inner surfaces of the mould prior to pouring to make sure the plaster slab could be easily removed after curing. The cast plaster slab was allowed to cure for 30 minutes in the mould and then it was taken out and placed in a room environment for the final curing process. As indicated in chapter three the flexural and compressive strength of the plaster is relatively stable after four weeks under room cure conditions, thus the plaster slabs were stored in the room environment for four weeks before being subjected to the test, except for one sample which had a shorter curing time in order to study the influence of rock strength on the behaviour of the TSL. Figure 4.28c shows the plaster slabs prepared for testing. The extensions on the edges of the slab were scraped off using a spatula. After two weeks of curing in the room environment, four holes were drilled through the slabs to accommodate the bolts (Figure 4.28d).

For the control sample, two layers of rubber sheet 600 mm in length and 200 mm in width were glued to the rear plaster slab to prevent spalling of plaster fragments (Figure 4.28e). The glue was allowed to cure for 24 hours and then four holes corresponding to the holes in the plaster slabs were drilled through the rubber mats. Three plaster slabs and the slab with the glued rubber mats were bolted together using bolts and plates. The last step of the sample preparation was to cap the sample to produce smooth and levelled surfaces at the top and bottom of the sample (Figure 4.28f), which helped to distribute the load evenly during the testing.
Welded steel mesh confined samples were prepared in a similar way to the control group samples except that 580 mm by 200 mm steel mesh sheets were attached to the plaster slabs using the bolts and steel plates, as per the control samples. The reason
why the mesh did not fully cover the plaster slab surface was to prevent the mesh being
directly loaded by the loading platen during the test.

Compared with steel mesh confined group sample, the preparation of the TSL
reinforced samples were more complicated. The difference lies in that the polymer
material with fibre reinforcement was bonded to the front plaster slab instead of just
attaching and fixing by plates and bolts as was the case in the mesh group. Note that
the procedures of bonding of the FRP B and FRP C were not identical due to the
different reinforcement mediums applied. While polymer B was reinforced with three
layers of glass fibre sheet, polymer C was reinforced with bar-chip fibre. In order to
bond the polymer onto the plaster slab, the slab was placed into a timber mould, and
grease was applied onto the inner surfaces of the mould to ensure the easy removal of
the sample after curing. For the FRP C group sample, the polymer was divided into
two equal batches. The first batch was poured onto the surface of the slab and then
distributed evenly by the use of a spatula (Figure 4.29a). Then, the bar-chip fibres were
applied uniformly on the polymer and they were gently pressed (Figure 4.29b). After
that, the other batch of polymer C was poured on top and distributed evenly (Figure
4.29c). Finally, a plaster cap wrapped with food wrap was placed onto the polymer to
produce a smooth surface (Figure 4.29d). The plaster slab with the bonded TSL was
allowed to cure for 24 hours in the mould. When it came to the FRP B confined group,
the polymer material was divided into four equal batches and glass fibre sheet was cut
into three 580 mm by 200 mm mats. One batch of polymer was poured and distributed
evenly, and then the first fibre mat was placed onto the polymer and gently rolled until
it was saturated with the polymer. This process was repeated till all four batches of
polymer material were applied. The following procedures were the same as applied in
bonding FRP C onto the plaster slab. After that, the TSL reinforced plaster slab was
bolted together with three other plaster slabs, including the one with rubber mats.
Prepared samples ready for testing are shown in Figure 4.30.
Figure 4.29 Preparation of the fibre reinforced polymer reinforced samples

Figure 4.30 Prepared samples ready for testing
4.4.3 Test set-up and procedure

Figure 4.31 describes the test set-up. The sample was placed on a spherical seat to achieve uniform load distribution during the test. The tests were conducted under displacement control mode, and the loading rates of the four groups of sample were different. For the control group, the loading rate was kept constant at 0.2 mm/min; for the steel mesh and TSL confined samples, the loading rate was initially set at 0.2 mm/min up to failure of the plaster slab and then increased to 1 mm/min up to the end of the test. The load and the displacement of the platen were recorded during the test. A laser displacement sensor was also employed to monitor the lateral deflection of the central point on the front surface of all samples.

![Test set-up](image)

Figure 4.31 Test set-up

4.4.4 Results and discussion

The load versus vertical displacement/lateral deflection curves of the samples are illustrated in Figure 4.32. It is clear from Figure 4.32 that, the control samples failed in a brittle way while the steel mesh confined sample was still able to resist a certain level of load after reaching the peak load. As expected, the TSL confined samples also exhibited residual strength.
Figure 4.32 Load vs vertical displacement/lateral deflection curves
It is apparent from the figure that the stiffness of TSL reinforced samples was very similar. Due to the fact that there was a 1 mm gap between the mesh and the surface of the front plaster slab for the steel mesh_2 sample, its stiffness was slightly smaller than that of steel mesh_1. A small variation in the stiffness also occurred between the two control samples. A possible reason could be that the four plaster slabs for sample two were not assembled as tightly as those for sample one, thus allowing slip along the bedding planes.

Figure 4.33 plots the typical load versus vertical displacement curves of samples in each group. It is apparent from the figure that the load increased slowly with displacement at first. This was attributed to the slack in the system being taken up, including any gap at the contact surfaces between two adjacent plaster slabs. As the load increased to a critical point, the slope of the curve increased significantly in all of the surface confined samples, but not as significantly in the control samples. This suggests that the mesh and TSL were now influencing the system stiffness. During this stage, lateral restraint was provided by the mesh and TSL to prevent the buckling of the plaster slabs, resulting in an increase in the stiffness of the samples.

Figure 4.33 Typical load versus vertical displacement curves of samples

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As the load kept growing, cracks initiated and the sample started softening. The load still increased with the displacement, however, due to crack developing and the sample softening induced by cracking, the load increase rate was not as great as before. This is clearly shown in Figure 4.34. When the peak load was reached, the confined samples and the unconfined samples performed significantly differently. With regard to the unconfined samples, as there was no confinement the load immediately dropped to near zero and the test was terminated. In contrast to the control samples, the steel mesh and TSLs confined samples were still able to provide resistance after a substantial fall in load. Note that there was no post failure behaviour recorded for steel mesh_1, this was because the test was terminated after the peak load was achieved.

![Figure 4.34 Typical load versus lateral deflection curves of samples](image-url)

4.4.4.1 Stiffness

In order to compare the steel mesh and TSLs in resisting the buckling of the samples, the stiffness of the samples was calculated. The overall stiffness and effective stiffness of the samples can be calculated using the following formulas:

\[ k_o = \frac{l_p}{d_p}, \quad k_e = \frac{l_p-l_c}{d_p-d_c} \]  \hspace{1cm} (6.1)

Where:
The stiffness of the samples are listed in Table 4.7. Due to the existence of the slack in the samples, the effective stiffness of the samples was greater than their overall stiffness. Note that as the plaster slabs of FRP C_3 were tightly assembled, its effective stiffness was the same as its overall stiffness. As expected, the effective stiffness of the two control samples were the lowest being 54 kN/mm and 48 kN/mm respectively. The confinement provided by the steel mesh and TSLs helped to improve the effective stiffness. To be specific, steel mesh_1 and steel mesh_2 had an effective stiffness of 98 kN/mm and 62 kN/mm respectively. The big difference in the stiffness between the two samples was due to the fact that while the steel mesh was closely attached to the plaster surface for steel mesh_1 sample there was an approximate 1 mm gap between the steel mesh and the plaster surface for steel mesh_2 sample. Steel mesh_1 was able to generate resistance force immediately after the buckling of the plaster. However, steel mesh_2 did not start resisting the buckling of the plaster slab until the mesh and the plaster came into contact. It is important to note that steel mesh_2 was more representative of an in situ scenario in underground strata control as the roadway excavation surface is not usually flat. The effective stiffness of FRP C_1 and FRP C_2 almost doubled that of the control sample, being 99 kN/mm and 121 kN/mm respectively. The significant variation in the effective stiffness for the two samples was a result of different plaster curing time. While the plaster slabs of the first sample were cured for four weeks those of the second sample were cured for only one week. It has already been shown in section 3.3 that plaster had greater UCS and flexural strength with longer curing period. Thus, the test results indicated that the FRP C was more effective than steel mesh in resisting the buckling of weak rock. The FRP B group samples had the greatest effective stiffness averaging 154 kN/mm, which was
202% greater than the unconfined sample. The FRP B confined samples had greater stiffness than the FRP C confined samples because FRP B was stiffer and stronger than FRP C, as discussed in section 3.4.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Overall stiffness (kN/mm)</th>
<th>Effective stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control_1</td>
<td>43</td>
<td>54</td>
</tr>
<tr>
<td>Control_2</td>
<td>32</td>
<td>48</td>
</tr>
<tr>
<td>Steel mesh_1</td>
<td>70</td>
<td>98</td>
</tr>
<tr>
<td>Steel mesh_2</td>
<td>48</td>
<td>62</td>
</tr>
<tr>
<td>FRP B_1</td>
<td>105</td>
<td>153</td>
</tr>
<tr>
<td>FRP B_2</td>
<td>109</td>
<td>154</td>
</tr>
<tr>
<td>FRP C_1</td>
<td>73</td>
<td>99</td>
</tr>
<tr>
<td>FRP C_2</td>
<td>80</td>
<td>121</td>
</tr>
<tr>
<td>FRP C_3</td>
<td>69</td>
<td>69</td>
</tr>
</tbody>
</table>

It is apparent from Table 4.7 that TSL confined samples had much greater stiffness than the second steel mesh confined sample, indicating the TSLs will be more effective than the steel mesh in preventing the buckling of rock strata in underground mines. This was because TSLs were intimately contacted with the plaster surface while the mesh was not. Test results also demonstrated that even if the mesh was closely attached to the plaster surface, the stiffness of TSL confined samples was still greater.

4.4.4.2 Strength

The strength of the buckling test samples are presented in Table 4.8. The two control samples had peak loads of 66 kN and 60 kN, averaging at 63 kN. As expected, the sample became stronger after being confined with steel mesh. To be specific, when the mesh was closely attached to the plaster slab surface (steel mesh_1), the confined sample was able to resist a load up to 128 kN almost double that of the control sample.
When there was an approximately 1 mm gap between the mesh and plaster slab (steel mesh_2), the peak load achieved was 94 kN. Even though the peak load was 49% greater than that of the control sample, it was 27% less than that of steel mesh_1. This was because the steel mesh was not able to confine the buckling plaster slabs until they deflected enough. The test results indicate that the gap between mesh and the excavation rock surface has a detrimental effect on the confinement ability of the steel mesh. As mentioned above, compared with steel mesh_1, steel mesh_2 is more representative of the in situ scenario.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Peak load (kN)</th>
<th>Cracking load (kN)</th>
<th>Residual strength (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control_1</td>
<td>66</td>
<td>57</td>
<td>0</td>
</tr>
<tr>
<td>Control_2</td>
<td>60</td>
<td>54</td>
<td>0</td>
</tr>
<tr>
<td>Steel mesh_1</td>
<td>128</td>
<td>92</td>
<td>--</td>
</tr>
<tr>
<td>Steel mesh_2</td>
<td>94</td>
<td>43</td>
<td>18</td>
</tr>
<tr>
<td>FRP B_1</td>
<td>198</td>
<td>101</td>
<td>33</td>
</tr>
<tr>
<td>FRP B_2</td>
<td>202</td>
<td>106</td>
<td>30</td>
</tr>
<tr>
<td>FRP C_1</td>
<td>147</td>
<td>102</td>
<td>9</td>
</tr>
<tr>
<td>FRP C_2</td>
<td>126</td>
<td>--</td>
<td>13</td>
</tr>
<tr>
<td>FRP C_3</td>
<td>140</td>
<td>--</td>
<td>18</td>
</tr>
</tbody>
</table>

Compared with the steel mesh, the two types of TSL material had a more positive effect on confining the buckling plaster slabs. The peak loads of FRP C_1 and FRP C_3 reached 147 kN and 140 kN respectively, with the average strength of 143 kN, 127% and 53% stronger than the control sample and steel mesh_2 respectively. The peak load achieved by FRP C_2 was 126 kN, less than that of the other two FRP C samples. This was as a result of the relatively shorter curing time for the plaster slabs. FRP B confined sample had the greatest peak load averaging at 200 kN, which was 218%, 113% and 40% stronger than that of the control sample, steel mesh confined
sample and FRP C confined sample respectively. The standard deviations of the peak loads achieved by the control samples, FRP B confined samples and FRP C confined samples were 5 kN, 3 kN and 5 kN respectively, which were very small compared with the peak loads of the three groups of sample. Thus, it can be concluded that the buckling test was repeatable.

The loads at which cracking of the plasters initiated are also listed in Table 4.8. It is apparent that the reinforced samples had much greater crack initiating load than the unreinforced samples except for steel mesh_2. As with stiffness and strength the low crack initiating load of steel mesh_2 was the result of the mesh not being closely attached to the plaster surface, thus being passive to initial displacement of the sample. While the average cracking load for the two control samples was 56 kN, the cracking load of steel mesh_1 reached 92 kN. The TSL reinforced samples were the strongest and did not crack until reaching a load of around 101 kN, which indicated the TSL was better than steel mesh in resisting cracking. Preventing the rock from cracking is desirable for underground rock support as it allows the rock to preserve its inherent strength.

Test results also demonstrated that while the unconfined samples failed in a brittle way the confined samples were still able to resist varying levels of load over a long displacement after reaching the peak load. Note that there was no residual strength recorded for steel mesh_1 as the test was terminated soon after the maximum load. The residual strength of FRP B confined samples was greater than that of FRP C confined samples, which was due to FRP B being stronger than FRP C. It is also worthwhile to note that FRP B confined samples had greater residual strength than steel mesh confined sample, being 32kN and 18 kN respectively.

4.4.4.3 Failure mode

Figure 4.35 shows the failure modes of the samples with and without confinement. The control sample failed around the centre area of the plaster slabs due to buckling.
During loading, tensile cracks initiated at the outer surface of the first slab. As the load increased the crack propagated through the slab and tensile cracks occurred at the centre of the three other slabs (Figure 4.35a). Steel mesh confined samples failed in a similar way to the control group, where a tensile crack started in the front slab and then developed in the three other slabs (Figure 4.35b). During further loading, additional cracks developed around the centre and the bolting areas. Note that the mesh did not break in the test.

![Figure 4.35 Failure modes of samples](image)

The TSL confined samples experienced a different failure mode from the control and mesh groups. A shear crack, instead of a tensile crack as was the case in the control and mesh confined samples, initiated at the centre area of the front slab and was then followed by tensile cracks at the centre region of the other slabs (Figure 4.35c&d). This was due to the confinement generated by the TSL materials, where the front slab experienced ‘triaxial’ loading conditions during the test. The mesh did not have this mechanism even though it was able to provide confinement. Both of the TSL confined
samples were capable of resisting further load after the development of cracks at the centre area. During the test, de-bonding of the bar-chip fibre from the polymer material was observed at the centre area of the FRP C sheet, which may have affected the residual strength. The FRP B sheet was cut off along the edge of the bolt bearing plate, which indicated that better performance would be achieved if the steel plates were rounded.

4.5 Thin spray-on liner and steel mesh in supporting strata with guttering
Guttering or cutter failure is one of the common roof failure modes in underground coal mines. Many factors such as high vertical stress, excess horizontal stress, relative stiffness between coal and its immediate roof, large topographic relief, bed separation and gas pressure and geological anomalies were reported to contribute to the formation of guttering (Su & Peng, 1986). Guttering in weak roof strata was found to form progressively in an underground coal mine (Zhang et al. 2007). Peng (2008) described the formation of guttering as cracks at the rib-roof corners firstly develop vertically till they encounter a competent stratum, and then the cracks will propagate horizontally along the separated bedding plane. Eventually, the roof strata below the bedding plane may fall down gradually or as a whole. According to Richmond et al. (1986), the guttering roof may be followed by cantilever and high arch failure and eventually results in a major roof fall. Gadde and Peng (2005) agreed with Richmond et al. (1986) and stated that roof guttering is sometimes a precursor to a major roof fall. TSLs, which are an innovative roadway support material to take the place of steel mesh, have to be able to support the guttering roof before application in underground coal mines. In this section, a large scale laboratory test was designed to compare a TSL material with steel mesh in supporting strata with guttering.

4.5.1 Experimental programme
In order to study the behaviour of a TSL and steel mesh in reinforcing strata with guttering, two large scale samples (TSL reinforced sample and steel mesh reinforced sample) were prepared. Figure 4.36 shows the schematic of the test. The guttering roof
was mimicked by many triangular concrete prisms. The prisms were layered and placed into a steel frame as shown in the figure. The steel frame consisted of three steel plates. The back steel plate acted as a competent stratum, the top and bottom plate were used to apply load to the sample. The steel mesh was attached to the front surface using bolt and plates, the bolts also went through the sample and were bolted at the back, the TSL material was bonded to the front surface of the prisms and bolted as per the mesh.

Figure 4.36 Schematic of the sample

4.5.2 Sample preparation

The aim of this experiment was to study the behaviour of a TSL and welded steel mesh in providing reinforcement to a rock mass prone to guttering. As implied above, tests were conducted on two samples named TSL confined sample and steel mesh confined sample respectively. Due to the availability of material at that time, polymer A was selected. Polymer A reinforced with three layers of glass fibre sheet was used to mimic the TSL. Figure 4.38 shows the sample preparation procedures. The first step to prepare the sample was to cast the triangular concrete prisms. The triangular prisms were 200 mm in width, 400 mm in length and 53 mm in height with acute angles of
28°. A timber mould (Figure 4.37a) was firstly placed on the floor, and then concrete was poured into it to prepare the prisms (Figure 4.37b). Oil was sprayed onto the surface of the mould prior to the pouring of the concrete to make the removal of the prisms easier. The prisms were allowed to cure in the mould for 24 hours and then they were taken out of the mould. 16 mm holes were drilled through some of the prisms so that threaded bar, to simulate rock bolts, could be grouted in (Figure 4.37c). The triangular prisms, 49 whole prisms and 14 half prisms, were layered into the steel frame to form a 400 mm × 400 mm × 800 mm rectangular concrete block. Welded steel mesh, 780 mm in length and 400 mm in width, was bolted to the concrete surface. The steel mesh was cut to size from mesh procured from a local coal mine. Figure 4.38(a) illustrates the prepared steel mesh confined sample.

Figure 4.37 Procedures of sample preparation
Bonding of the 5 mm thick fibre reinforced polymer onto the concrete block followed the following steps: an initial coat of polymer was poured onto the concrete surface to form a bond to the concrete, and then a glass fibre sheet cut to size was placed on top and rolled into the polymer. This process was repeated twice with a final cover of polymer rolled into the last fibre sheet. Note that the polymer and the fibre did not fully cover the concrete surface. This was done on purpose to guarantee that the
polymer sheet did not touch the top or bottom platen during the loading process so that it was not loaded axially. The concrete blocks were allowed to cure for at least four weeks before the polymer was applied, which in turn was left to cure for two weeks.

Figure 4.38 Samples ready for testing: (a) steel mesh confined sample and (b) fibre reinforced polymer confined sample

4.5.3 Test set-up and procedure

Figure 4.39 illustrates the test set-up. The samples were placed onto a spherical seat to ensure the load was uniformly applied to the sample during the test process. The tests were performed in a 500 t Avery compression testing machine. Displacement control mode was selected and the loading rate was 0.5 mm/min. The load applied, the displacement of the loading platen and the deformation of the centre point of the sample front surface were recorded by a 5000 kN load cell, a LVDT and a laser respectively.
4.5.4 Results and discussion

The sample was forced to expand laterally as it was compressed at the top and bottom surface during the test on the steel mesh reinforced sample. Several triangular concrete blocks were observed to slip or move after only limited displacement, again illustrating the passive nature of steel mesh when used for skin confinement. The test terminated when the front surface of the sample was so close to the laser that it may touch if the test continued. Note that the steel mesh did not fail at this stage. Block slippage or movement was also found in the test on the TSL reinforced sample, but it did not move or slip as much as in the other test. De-bonding of the TSL sheet to the triangular prisms was observed in the test. One of the bolts broke during the loading process and the test was stopped for safety reason. As before, this test was halted without failure of the fibre reinforced polymer sheet.

The load versus lateral deflection behaviours of the two samples are illustrated in Figure 4.40, the sudden drop in the TSL graph is a result of the failure of the bolts. Load fluctuations can be found in both graphs, which was a result of block slippage during the test. It is obvious that there are not as many fluctuations in the TSL graph, indicating that blocks did not slip to the same extent as in the steel mesh reinforced sample. The reason for this occurrence was that the adhesion at the interface of the concrete prism and the TSL restricted the movement of the blocks, while there was no
adhesion between the steel mesh and concrete. It can be seen from the figure that the two samples behave similarly up to around 40 mm deflection, however, the TSL reinforced sample had a much stiffer load versus lateral deflection relationship after 40 mm deflection. Figure 4.41 shows the load versus vertical displacement curves of the two tests, which also confirmed that the TSL sample was stiffer than the steel mesh. As neither the TSL sheet nor the steel mesh broke during the test, there was no point to compare the maximum loads achieved in the two tests.

![Figure 4.40 Load vs lateral deflection curves](image1)

![Figure 4.41 Load vs vertical displacement curves](image2)
It is worthwhile to note that the load of the steel mesh reinforced sample at a deflection of 80 mm is around 160 kN, while the corresponding value of the TSL sample is approximately 450 kN. This illustrates how the TSL would work as a composite with the rock mass to help the rock mass to maintain its integrity. Figure 4.42 shows the displacement of each sample after 80 mm deflection. It is obvious from this figure that the TSL restricts the development of guttering significantly better than steel mesh, a result of the bonded TSL acting as a composite with the substrate to immediately limit deformation and assist the strata to maintain integrity whilst substantial deformation is occurring.

Figure 4.42 State of guttering for the two tests after 80 mm deflection

The large scale guttering tests show that a 5 mm thick TSL was better than steel mesh in terms of restricting displacement of the concrete blocks. This was because the TSL was bonded to the substrate and any movement of the concrete was able to trigger an immediate reaction in the TSL. This, however, was not the case when it came to steel mesh. It is common knowledge that a major principle of rock support is to help the rock mass to maintain its integrity and self-support. In underground coal mines, formation of mining induced rock fractures cannot be prevented. The fractured rock mass may suffer bulking due to fracture displacement if there is no confinement stress.
present. The application of a TSL at an early stage minimises the displacement of fractured rock skin, which in turn helps the rock to preserve its ability to self-support. It may be argued that the load versus lateral deflection graph of the TSL was not much stiffer than that of steel mesh before deforming 40 mm. The reason for this phenomenon was that gaps existed between the surfaces of the concrete prisms that were slowly closed during the loading stage, however, it was interesting to note that the load versus displacement curve of the TSL reinforced sample was always stiffer than that of the steel mesh reinforced sample.

4.6 Summary and conclusions

A series of small scale laboratory tests were conducted to investigate the beam enhancement capacity of a thin spray-on liner (TSL). The results from the laboratory tests clearly demonstrated that the polymer based TSL reinforcement significantly increased the flexural strength of the beams in all cases. The tests indicated that a TSL bonded to the plaster surface more than doubles the sample strength. To gain a better understanding of the mechanism by which thin spray-on liners (TSLs) help to stabilise the ‘cracked’ roof or walls of an underground tunnel, a series of beams with different notch shapes and notch reinforcement conditions were tested to failure in a four-point bending test. It was found that the TSL coated notched samples were approximately 14 times stronger than the notched plaster sample without the polymer coating. An increase in the strength of the TSL coated sample with a notch was observed when the notch was filled with polymer. This illustrated the benefits of TSL propagation into the cracks and joints, however, even if the polymer is unable to penetrate into the cracks and joints, the TSL composite still provides a substantial level of reinforcement to fractured strata skin. Results in all cases also showed that failure at the crack tip had been resisted and that failure of the beam initiated elsewhere, which indicated that the application of the liner to the face of notched beams causes the stress concentration at the crack tip to be redistributed elsewhere. This ability of the thin polymeric liner to resist crack propagation suggests that the polymer acts as a composite with the beam, a relationship which traditional steel mesh does not have. It can be concluded from the laboratory test that substantial reinforcement of the rock substrate in an underground
tunnel is possible with the application of a polymeric TSL, even if the rock surface is cracked or fractured. The tests also indicated, that surface bonding and penetration of the polymer into the fractured strata enables the polymer skin to act as a composite material with the immediate substrate, helping the roof to maintain its integrity and resisting crack propagation into the surrounding strata.

A series of large scale laboratory test were performed to investigate the behaviour of TSLs and steel mesh in supporting strata with weak bedding planes, buckling strata and strata with guttering. Results of the test indicated that fibre reinforced polymer showed good performance in reinforcing strata with weak bedding planes. It was able to significantly enhance the strength of the strata with weak bedding planes. As one of the rock bolts in the steel mesh confined sample broke a direct comparison with the TSL confined sample was not possible, however, the test results clearly showed that the TSL was more effective over steel mesh in restricting softening of fractured strata. In addition, a better performance would be expected if a more rounded steel plate was employed in the test as the TSL only failed along the sharp edge of the plates.

Both types of TSL material (fibre reinforced polymer B and fibre reinforced polymer C) had better performance over steel mesh in supporting buckling strata. The TSL confined samples had greater peak load than the mesh confined samples. In addition, they were generally stiffer than the mesh samples, which is a desirable feature for underground rock support as a stiffer support medium is more effective in restraining rock dilation and ultimately helping to preserve the inherent strength of the strata. Test results indicated that the gap between the mesh and the rock had a significant adverse influence on the support performance of the mesh, however, this would not be a problem for the TSLs as they are bonded directly to the rock strata. In contrast to the mesh confined samples, TSLs confined samples had greater crack initiating load. This indicated that TSLs were better than mesh in terms of preventing the buckling rock from cracking and potentially preserving the inherent strength of the rock, which is beneficial to the rock stability.
The maximum loads for the TSL (fibre reinforced polymer A) and steel mesh reinforced strata with guttering were not achieved in the laboratory testing. Compared with steel mesh the TSL demonstrated obviously stiffer supporting performance, which is desirable for guttering roof support as it enables the fractured rock to preserve its self-support ability and eventually contributes to preventing guttering development. The following chapter discuss the load bearing capacities of steel mesh and TSLs.
5 INVESTIGATION INTO THE LOAD BEARING CAPACITIES OF THIN SPRAY-ON LINERS AND WELDED STEEL MESH

5.1 Introduction
Welded steel mesh, being a traditional surface support component, has been used successfully in underground coal mines to help control the roof and rib for many years (Nemcik et al. 2009). It normally consists of longitudinal wires with transverse wires welded to them, it is usually applied in underground mines together with rock bolts or cable bolts. It is common knowledge that it is not practical to prevent mining induced fractures from forming but it is possible to enhance the excavation surface condition by applying a support system at an early stage. The objective of rock support in this case is to preserve the rock’s self-supporting ability by limiting the movement of key blocks rather than attempting to hold the dead weight of the loosened rock. Although mesh is a passive support material, it is able to prevent broken rock from falling down and provides confinement to the unstable rock mass.

Many researchers (Tannant 2004a; Thompson 2004; Dolinar 2006; Dolinar 2009) conducted tests on welded wire mesh in the laboratory, but almost all the meshes tested were within the size of 1.5 m by 1.5 m. It was shown that smaller mesh sheets exhibited different deformation characteristics than the larger mesh sheets in laboratory tests (Tannant 2001) and numerical modelling (Gadde, Rusnak & Honse 2006). In order to accurately evaluate the performance of mesh and subsequently compare with thin spray-on liners (TSLs), full size mesh sheets were tested in this study.

TSLs have been considered as an alternative to welded steel mesh for a long time. As listed in chapter two, a wide range of laboratory tests have been performed to investigate the mechanical properties of TSLs, however, most of the samples involved were relatively small, and the data on full size laboratory tests on TSLs are limited. As such, this chapter discusses the behaviour of full size TSLs subjected to push loading
in the laboratory. Note that full size TSLs indicated that the TSL size was larger than 1 m × 1 m which was big enough to accommodate four adjacent rock bolts as installed in situ. In addition, a full size thin spray-on liner (TSL) sheet adhered to concrete blocks was prepared and tested to investigate the behaviour of a TSL acting as a composite with the concrete blocks.

5.2 Evaluation of the load bearing capacity of welded steel mesh

The test apparatus included three parts: strong floor, loading frame and the control unit. Figure 5.1 shows the schematic of the strong floor. The strong floor consisted of thick steel plates mounted onto a concrete base with grooves between two adjacent plates. While the gap between the two adjacent steel plates is 50 mm the groove below the steel plates is 100 mm, this allows the bolt holder to be tightly held when subject to pull loading.

As shown in Figure 5.2, the loading frame was composed of a steel frame, bolted onto the strong floor, with a hydraulic ram mounted on it. A load cell was connected to the hydraulic ram. The control unit comprised a computer which controlled the testing process and a data logger for data acquisition.
5.2.1 Sample preparation

Both roof mesh and rib mesh commonly used in Australian coal mines were subjected to a full scale pull test. Four sheets of roof mesh and one sheet of rib mesh were prepared for testing. The roof mesh was 1.35 m by 3.6 m and the rib mesh was 1.5 m by 4 m. The roof mesh was fabricated from 5 mm diameter longitudinal and transverse steel wires with 7 mm diameter longitudinal reinforcing wires located where rock bolts would be installed and where the mesh overlaps the next sheet. The rib mesh consisted of 4 mm diameter steel wires without reinforcing steel wires.

5.2.2 Test set-up

Figure 5.3 shows the full scale pull test set-up. The mesh was bolted to the strong floor with a bolt spacing of 1 m by 1 m. In order to minimise slippage of the mesh, a torque wrench was used to apply 240 Nm of torque to the bolts to provide a consistent pretension force, timbers were also placed between the bolts in the same groove to prevent lateral slip. In order to more closely simulate loading in an underground coal mine, the load was applied by pulling a spherical seat, instead of the usual square plate, through the mesh. The spherical seat had a diameter of 300 mm, and a dome height of 41 mm.
The test was run in displacement control mode with a loading rate of 24 mm/min, which was slow enough to simulate static loading. During the tests, the load was measured by a 100 kN load cell with an accuracy of ±0.2 kN and the displacement was monitored by a linear variable differential transducer (LVDT) with an accuracy of ±0.6 mm. The load and displacement were recorded during the tests.

5.2.3 Results and discussion

The load versus displacement curves of all the tests are shown in Figure 5.4. As shown in the figure, there were many ‘saw tooth’ in the curves. The ‘saw tooth’ was caused by slippage of the mesh underneath the rock bolt plate. The big load drops in the curve were due to wire failure of the mesh or, in the case of test 1, lateral mesh slippage. Note that timber was not applied in the first test, where it was observed that the bolt holders moved along the groove under load, which resulted in excess slippage. As such, timbers were applied in the following tests.
The maximum load and corresponding displacement of the tests are listed in Table 5.1. It is apparent from the table that the roof mesh with thicker wires displayed greater peak load capacities compared with the rib mesh with thinner wires. Specifically, the
peak load of roof mesh_2 was 48 kN, but the peak load of rib mesh_1 was only 21 kN, less than half of that of roof mesh_2. It is interesting to note that while the difference in peak load for the two mesh types is significant the diversity in displacement at peak load for them is not so remarkable. The peak load achieved by roof mesh_1 was 44 kN, which was around 5 kN smaller than that of the other three roof mesh samples. This resulted from the fact that timber was not placed between the bolt holders during the test. Without the restraint from the timber, the bolt holders moved along the groove, resulting in uneven loading at the bolts. As such, the peak load achieved by roof mesh_1 was lower.

<table>
<thead>
<tr>
<th></th>
<th>Maximum load (kN)</th>
<th>Displacement at maximum load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof mesh_1</td>
<td>44</td>
<td>466</td>
</tr>
<tr>
<td>Roof mesh_2</td>
<td>48</td>
<td>464</td>
</tr>
<tr>
<td>Roof mesh_3</td>
<td>51</td>
<td>490</td>
</tr>
<tr>
<td>Roof mesh_4</td>
<td>48</td>
<td>502</td>
</tr>
<tr>
<td>Rib mesh_1</td>
<td>21</td>
<td>485</td>
</tr>
</tbody>
</table>

Figure 5.5 illustrates the typical load versus displacement curves of a roof mesh and the rib mesh. It is worthwhile to notice that the roof mesh generally had slightly stiffer initial load-displacement response than the rib mesh before they experienced a displacement of approximately 310 mm, after which wire failure or slippage occurred in the rib mesh while the roof mesh can still bear an increasing load. The in situ support capacity of the mesh is mainly dependent on the rocks self-support ability and the strength of the mesh wires. It can be concluded that the roof mesh was better than the rib mesh for rock support as it had a stiffer initial load-displacement response, which means it can prevent rock unravelling at less displacement and help to maintain the self-support ability of the rock mass, the individual wires of roof mesh could also accept a higher load before failure.
Another important phenomenon observed during the tests is that almost all of the wire failure of roof mesh occurred near the loading dome, however, all the wire failure for the rib mesh occurred near the load bearing plates (Figure 5.6). This indicated the importance of the reinforcing wire in the roof mesh.
5.3 Evaluation of the load bearing capacities of thin spray-on liners

As a traditional surface support, welded steel mesh has been successfully utilised in underground coal mines as a skin confinement medium for roof and rib strata for many years. It is, however, difficult to automate the installation process, thus it is both time consuming and labour intensive. Moreover, steel mesh is a passive support and does not provide surface confinement until substantial rock displacement occurs. To meet the roadway development requirements of future longwalls, the coal industry requires a significant increase in roadway development rates over those currently achieved. TSLs are an innovative rock support material which can be applied automatically so that increased roadway development rates can be achieved. In addition, they have many other merits over steel mesh, for example, they can be applied remotely to improve personnel safety and they bond to the rock surface generating resistance to rock displacement immediately after application.

Prior to the application of TSLs into coal mine roadway reinforcement, it is essential to firstly evaluate their load bearing capacities in the laboratory. As such, this section focuses on an experimental investigation of the behaviour of full size TSL sheets and a TSL-concrete composite subjected to push loading.

5.3.1 Test apparatus

The test apparatus included the strong floor, loading frame and control unit as described previously, and a specifically fabricated steel frame as shown in Figure 5.7. The steel frame consisted of three parts: steel base channel, steel spanning channel and steel fastening angle. There were four steel base channels which were bolted together to form a square fabrication. The TSL sheet was placed onto the base channels, and it was fastened by bolting the four fastening angles and two spanning steel channels onto the base channels. Four extra holes which were 1 m apart were drilled through the two spanning channels and were used to bolt the TSL sheet to the spanning channels. This was to simulate the in situ case. During the test, the steel frame was bolted onto
another two steel channels which were bolted onto the strong floor by the use of bolts and bolt holders.

Note: 1- steel base channel, 2- steel spanning channel, 3- steel fastening angle, 4-hole

Figure 5.7 Steel frame specifically fabricated for the test

5.3.2 Sample preparation

The polymer used in the tests was not the actual thin spray-on liner being formulated at the University of Wollongong, but just an off the shelf product used to validate the test procedures. Four 1.4 m by 1.4 m TSL sheets were prepared for the full scale laboratory test, one was fibre reinforced polymer (FRP) A and the other three were fibre reinforced polymer (FRP) B. Polymer A sheet was reinforced with three layers of glass fibre sheet. One polymer B sheet was reinforced with two layers of glass fibre sheet, and the other two polymer B sheets were reinforced with three layers of glass fibre sheet.

Figure 5.8 shows the sample preparation procedures for the full size FRP sheet. Four aluminium angles of 1.4 m in length were bolted onto a piece of plywood to create a mould for the preparation of the sample (Figure 5.8a). The mould was firstly placed
on the table and waxed 7 times, which was to ensure the easy removal of the TSL sheet after curing. It was also important to level the mould so as to help distribute the polymer material evenly.

Figure 5.8 Preparation of a fibre reinforced polymer sheet
Taking the preparation of polymer B reinforced with three layers of fibre glass sheet as an example. As the width of the glass fibre sheet was only 1.1 m which was 0.3 m shorter than the required size, each fibre layer was composed of two fibre sheets of 1.1 m by 1.4 m and 0.305 m by 1.4 m respectively, which allowed for a 5 mm overlap. The polymer was divided into four equal batches, then the first batch was poured into the mould and distributed evenly using a spatula (Figure 5.8b). After that, the first fibre mat was placed on top of the polymer and rolled into it gently (Figure 5.8c). This process was repeated twice with a final cover of polymer (Figure 5.8d). The sample was allowed to cure in the mould for 24 hours and then it was removed and further cured in the room environment for at least 3 days before the testing. Figure 5.8e shows a TSL sheet with the drilled holes, which was ready to be fixed to the steel frame. It is important to note that although the fibre reinforced polymer sheet was designed to be 5 mm in thickness, the actual thickness was 4.5 mm. The other TSL sheets were prepared in the same way.

One of the advantages that TSLs have over welded steel mesh in underground rock support is that a TSL can be bonded to the rock so as to act as a composite with the rock. In order to investigate the behaviour of a TSL in this situation, a FRP sheet with bonded concrete blocks was prepared and subject to the push loading. The sample consisted of three components: FRP B sheet, broken concrete blocks and ballast. The ballast was used to help distribute the concentrated load of the platen over the concrete blocks and ultimately the polymer sheet. This simulated the loading conditions expected in a coal mine roof.

In order to make the broken concrete blocks, concrete slabs were firstly cast using sand, cement and water (Figure 5.9a). The three materials were mixed with the weight percentage of 69%, 21% and 10% respectively. Grooves were cut into the concrete slab after curing for two days, which was to ease the breaking of the concrete slab. The grooves also divided the slab into many sections and each section was numbered for easy assembly in a later phase (Figure 5.9b). The slab was then broken into
concrete blocks along the grooves (Figure 5.9c). The concrete blocks were re-assembled following the numbers previously marked (Figure 5.9d).

Figure 5.9 Preparation of the concrete blocks

The next step was to bond the concrete blocks to the TSL sheet. As the concrete blocks were over 200 kg, it would have been difficult to put the TSL sheet with concrete onto the steel frame, therefore, it was decided to fasten the TSL sheet onto the steel base channels first and then bond the concrete blocks onto it. This process was completed in the following steps. Firstly, the TSL sheet was placed onto the steel base channels and was marked at the positions corresponding to the holes in the channels (Figure 5.10a). Note that marks corresponding to the four holes in the two steel spanning channels were also made on the TSL sheet. The four holes were 1 m apart and were used to accommodate the rock bolts applied to fix the bearing plates underneath the TSL sheet, which was to simulate the in situ application of TSLs in underground mines. Holes were then drilled through the TSL sheet at the marked positions. Secondly, the
TSL sheet was fixed onto the base channels by the use of fastening angles and the spanning channels (Figure 5.10b). Finally, the concrete blocks were bonded to the TSL sheet at the pre-determined area with a thin layer of polymer (Figure 5.10c). Figure 5.10d shows the concrete blocks bonded in place. The sample was allowed to cure for at least 24 hours before the final step.

![Figure 5.10 Bonding concrete blocks to the TSL sheet](image)

The final step was to add a 100 mm thick layer of ballast onto the concrete blocks, to help distribute the load evenly during testing, as mentioned previously. As the fastening angles were 75 mm high which were not high enough to confine the ballast, two timber plates were inserted into the gaps between the concrete block and the angles. A rubber mat cut to size with a thickness of 10 mm was placed between the concrete blocks and the ballast, which was to prevent the ballast falling onto the TSL sheet through the gaps between the concrete blocks. The ballast normally has sharp
edges which may lead to premature failure. Figure 5.11 shows the prepared sample ready for the testing.

5.3.3 Test set-up

The test set-up is illustrated in Figure 5.11. The TSL sheet was loaded to failure by pushing a spherical steel seat downwards. Displacement control model was chosen in this experiment, and the loading rate was set at 6 mm/min. The load and displacement were recorded during the test.

![Test set-up](image)

**Figure 5.11 Test set-up**

5.3.4 Results and discussion

The load versus displacement curves of the tests are illustrated in Figure 5.12. It is apparent that the samples experienced brittle failure but all of them showed various levels of residual strength. The post failure residual strength of TSL is desirable for underground rock support as it will still be able to provide confinement to the strata after failure. Table 5.2 lists the details of the test results. As expected, the TSL with more fibre sheets was stronger. Polymer B reinforced with two layers of fibre sheet (FRPB_2S) had a peak strength of 16 kN, while the maximum load increased to 31 kN when the polymer was reinforced with 3 layers of fibre sheet, almost double the strength of polymer B with 2 layers of fibre sheet. The maximum load achieved by polymer A reinforced with 3 layers of fibre sheet (FRPA_3S) was lower than that of polymer B with 3 layers of fibre sheet (FRPB_3S), being 26 kN. This result was in
agreement with the previous finding that the former had lower tensile and flexural strength than the latter. It is worthwhile to note that the polymer B reinforced with 3 layers of fibre sheet-concrete composite (FRPB_3S-concrete) had the greatest maximum load of 51 kN, which was 65% and 219% greater than that of the polymer B with 3 layers of fibre sheet and polymer B with 2 layers of fibre sheet respectively.

![Load versus displacement curves of the tests](image)

**Figure 5.12 Load versus displacement curves of the tests**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Maximum Load (kN)</th>
<th>Displacement at maximum load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FRPA_3S</td>
<td>26</td>
<td>83</td>
</tr>
<tr>
<td>FRPB_2S</td>
<td>16</td>
<td>61</td>
</tr>
<tr>
<td>FRPB_3S</td>
<td>31</td>
<td>72</td>
</tr>
<tr>
<td>FRPB_3S-concrete</td>
<td>51</td>
<td>105</td>
</tr>
</tbody>
</table>
Figure 5.13 compares the load-displacement curves of all the samples. It is clear that polymer B with 3 layers of fibre sheet was stiffer than that of polymer B with 2 layers of fibre sheet and polymer A with 3 layers of fibre sheet. This was because polymer B with 3 layers of fibre sheet had greater flexural modulus than the other two samples. As shown in section 3.4, the flexural modulus of the three samples was 3.8GPa, 3.4 GPa and 2.3 GPa respectively. Due to the existence of the ballast, the FRP-concrete composite was not as stiff as polymer B with 3 layers of fibre sheets, however, it was still stiffer than the other two samples.

The tests also indicated that the fibre sheet helped to improve the post failure behaviour of the samples. While there was almost no residual strength for polymer B with 2 layers of fibre sheet, polymer B with 3 layers of fibre sheets had a residual strength of approximately 8 kN over a displacement of around 16 mm. The FRP B samples were stiffer than FRP A sample, however, the post failure behaviour of FRP A was more desirable for underground rock support. After reaching the peak load of 26 kN, the load resisting capacity of FRP A dropped remarkably to about 8 kN at first and then experienced a slow increase to 10 kN over a displacement of about 32 mm. After that the residual strength of FRP A decreased in a stairs-shaped manner till the complete failure of the sample. The post-failure behaviour of the FRP-concrete composite was similar to that of FRP A and can be divided into three stages. In the first stage, the residual strength increased from approximately 20 kN to 22 kN over a displacement of about 12 mm and then fell to around 17 kN. In stage two, the residual strength stayed constant at around 17 kN and then decreased to about 8 kN. In stage three, the residual strength grew gradually from 8 kN to 13 kN over a displacement of 22 mm and then declined to around 3.4 kN. At this point the sample had reached the limit of deflection of the test rig.
The failure modes of the FRP B and the FRP-concrete composite were different. As shown in Figure 5.14, the plain FRP B sheet started to crack beneath the loading dome and then the crack propagated toward the edges. When the crack was long enough and could not resist any load the test terminated. With respect to the FRP-concrete composite, the crack initiated along one of the load bearing plates and then propagated to the edges. As the load kept increasing, the crack developed along the steel channel till the complete break of two sides of the TSL sheet. This suggests that the design of rockbolt bearing plates should avoid shard edges impacting on the TSL. This was supported by tests on bearing plate design (Nemcik et al. 2011b).
5.4 Comparing the load bearing capacities of thin spray-on liners and welded steel mesh

Full scale laboratory tests were conducted to enable comparison between the behaviour of steel mesh and thin spray-on liners (TSLs) in supporting broken strata. In particular it would allow comparison between the passive support behaviour of steel mesh and the composite behaviour afforded by a TSL.

Figure 5.15 shows the load-displacement curves of rib mesh, roof mesh, TSLs and the TSL-concrete composite. It is apparent that the TSLs were much stiffer than both the rib mesh and roof mesh. To be specific, while the displacement at peak load for both types of steel mesh was around 480 mm, the displacement at peak load for the TSLs and TSL-concrete composite was in the range of 60 mm to 105 mm. In order to generate a resistance force of 10 kN, the polymer B reinforced with 3 layers of fibre sheet and FRPB_3S-concrete composite experienced a displacement of 41 mm and 47 mm respectively, however, the rib mesh and roof mesh underwent a much greater displacement of 267 mm and 220 mm respectively. When the resisting force increased to 20 kN, the displacement for the plain TSL sheet and TSL-concrete composite had an increase of 18 mm and 17 mm respectively, reaching 59 mm and 64 mm. The displacement of the rib mesh and roof mesh rose to 436 mm and 358 mm respectively, experiencing a significant increase of 169 mm and 138 mm. As the resisting force kept
increasing, the difference in the displacement between TSL-concrete composite and the roof mesh kept growing. These results clearly indicated that TSLs were able to provide greater resisting force than steel mesh at the same rock displacement. Thus, it can be concluded that TSLs are superior in underground rock support as they are better than steel mesh in preventing rock dilating or displacing, which helps to maintain the inherent strength of the rock mass and ultimately is beneficial to the stability of roadways of underground coal mines.

![Figure 5.15 Load-displacement curves of steel mesh and TSLs](image)

Test results also showed that although the TSL sheets were not as strong as the roof mesh, most of them were stronger than the rib mesh. Specifically, the peak load of the rib mesh was 21 kN. Polymer A and polymer B reinforced with 3 layers of fibre sheet had greater peak load of 26 kN and 31 kN respectively. The roof mesh was much stronger and the peak load reached 51 kN which was 65% greater than that of the FRP B, however, when the FRP B was bonded to the concrete and formed a TSL-concrete composite, its load bearing capacity increased significantly to 51 kN, which was similar to that of the roof mesh. As such, it can be concluded that the TSL has the potential to take the place of steel mesh in terms of load bearing capacity.
It is interesting to note that all of the samples exhibited various degree of post failure behaviour. The residual strength of TSLs and TSL-concrete composite was much lower than their peak loads. In contrast, the initial post failure strength of rib mesh and roof mesh was similar to the ultimate strength. This was due to redistribution of load from a failed strand to another part of the mesh.

5.5 Summary and conclusions

The objective of this study was to determine the ultimate strength of thin spray-on liners (TSLs) with or without concrete blocks bonded and steel mesh using full scale tests and to compare the results. A sheet of rib mesh, four sheets of roof mesh, three types of fibre reinforced polymer (FRP) sheet and a FRP-concrete composite were subjected to full scale laboratory testing. Test results indicated that although the 4.5 mm thick plain TSL was not as strong as the roof mesh, it was stronger than the rib mesh. In addition, it was much stiffer, providing confinement at an earlier stage. The most important property of the TSL is the ability to bond to the rock surface, creating a polymer-rock composite. As with all composite materials, the whole system is greater than the sum of the parts. When comparing the concrete bonded TSL and the roof mesh it is evident that the TSL-concrete composite provided higher support loads at lower displacements while it continued to support the strata at large displacements. It can be envisaged that the TSL provides greater confinement to the strata at lower displacements and when overloaded it will still able to provide significant support to the fractured strata as displacement increases.

It is not practical to prevent the formation of mining induced fractures but it is possible to enhance the excavation surface condition by applying an effective support system at an early stage of mining. The nature of rock support is to preserve the rocks self-supporting ability by the use of the rock support material rather than holding the dead weight of the rock. Even if the plain TSL sheet was weaker than steel mesh, it was able to provide better load bearing capacity when bonded to the broken concrete slab than steel mesh in a similar situation. This suggests that the TSL shows superior potential for underground rock support.
6 NUMERICAL STUDY

6.1 Introduction
Conducting in situ tests to investigate the coal mine roof reinforcing mechanisms of polymer-based thin spray-on liners (TSLs) as an alternative to the passive support of steel mesh, currently used in underground coal mines, is both costly and time consuming and has potential occupational health and safety implications. As such, it was decided to develop numerical models to help further understanding of the TSL interaction with the immediate roadway roof. In addition, numerical modelling was also performed to validate the full scale laboratory tests of steel mesh.

6.2 Numerical modelling of the steel mesh behaviour

6.2.1 Numerical modelling of steel strands subjected to three-point bending test
The roof mesh subjected to the full scale laboratory tests in this study consisted of two different steel wires 5 mm and 7 mm in diameter. The performance of the two types of steel wire in the three-point bending test was firstly investigated using numerical modelling. The three-point bending test on the wires was previously described in section 3.5.2. The software used was Fast Lagrangian Analysis of Continua in 3 Dimensions (FLAC$^3$D), which is an explicit finite difference program. The single steel wire was simulated using the beam structural element in FLAC$^3$D. Boundary conditions corresponding to the physical test were imposed on the nodes of the beam structural elements: namely, no translation in the y-direction and no rotation about the x- and z-axes. The input parameters for the steel strands are listed in Table 6.1.

<table>
<thead>
<tr>
<th>Steel Strand Diameter (mm)</th>
<th>Young’s Modulus (GPa)</th>
<th>Poisson’s ratio</th>
<th>Plastic moment (Nm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>201</td>
<td>0.3</td>
<td>14</td>
</tr>
<tr>
<td>7</td>
<td>213</td>
<td>0.3</td>
<td>45.5</td>
</tr>
</tbody>
</table>
Figure 6.1 compares the laboratory test results and numerical modelling results. It is clear from the figure that the two results matched well, which indicates FLAC\textsuperscript{3D} is capable of evaluating the behaviour of welded steel mesh in a full scale pull test.

![Figure 6.1 Laboratory test versus numerical modelling](image)

6.2.2 Numerical modelling of steel mesh sheet subjected to a full scale pull test

The behaviour of the roof mesh in the full scale pull test was studied using FLAC\textsuperscript{3D}. The single steel wire subjected to bending was successfully simulated using the beam structural element, and therefore the welded steel mesh sheet was also modelled as a collection of beam structural elements with links corresponding to the weld points in the mesh. Welded steel mesh is usually used together with rock bolts in underground coal mines. Due to the strata movement, the load applied to the mesh is gradually transferred to the mesh region near the loading area and eventually to the rock bolt. The load transfer mechanism is closely related to the weld strength of wires. Of the eight weld shear tests conducted in this study none of them failed due to weld failure, demonstrating that the weld of the mesh was of good quality. This was confirmed by the laboratory tests where no weld failure in the pull tests occurred during testing of the full scale welded steel mesh. For this reason, weld failure was not considered in the numerical modelling. The slip along the steel mesh and strata contact was not
considered in the modelling as it is difficult to obtain the related input parameters. The dome loading was simulated by increasing the number of the loading nodes as the displacement increased. The material properties used in the model were the same as those employed in the modelling of the steel wire bending test. The modelling was only conducted to the point in which the first wire broke as the input parameters after this point were not available. Figure 6.2 illustrates the configuration of the steel mesh in the numerical model.

![Figure 6.2 Configuration of the steel mesh in the model](image)

Figure 6.3 shows the z-direction displacement of the mesh when the first wire broke. The load-displacement curve derived from the numerical modelling was compared with that from one of the laboratory tests in Figure 6.4. It was evident that the results matched well. Specifically, the modelled load at the first wire failure was 41 kN with a corresponding displacement of 460 mm. The two corresponding values in the laboratory test were 42 kN and 456 mm. The modelled wire mesh stiffness appeared greater than that of laboratory test, which may have been due to the fact that no slip was modelled. Some slip did occur in the physical test even though a high torque was applied to the bolts to prevent the mesh from slipping.
Figure 6.3 $z$-displacement of the mesh
Figure 6.4 Comparison of the load versus displacement curves between laboratory test and numerical modelling

In order to study the influence of the loading plate shape on the behaviour of welded steel mesh, subsequent modelling simulated loading using a flat plate. The load-displacement curves of the two models are illustrated in Figure 6.5. It is clear that there was not much difference between the two models except that the flat plate loading produced a slightly higher load at first wire failure and slightly stiffer initial load-displacement behaviour. This was because in the initial period the load applied was carried by more wires in flat plate loading, and the load was transferred to the bolt earlier.

Figure 6.5 Comparison of the load versus displacement curves between dome loading and flat plate loading
6.3 Numerical modelling of a thin spray-on liner for underground coal mine roof support

The behaviour of a thin spray-on liner (TSL) supporting an underground coal mine roof was studied using a FLAC\textsuperscript{3D} numerical model. FLAC\textsuperscript{3D} is able to analyse the behaviour of three dimensional structures built of soil, rock or other materials. In addition, FISH, which is a programming language embedded within FLAC\textsuperscript{3D}, can be used to define new variables and functions, making FLAC\textsuperscript{3D} a powerful analysis tool to deal with three dimensional geotechnical problems.

6.3.1 Description of the FLAC\textsuperscript{3D} numerical model

The ability of a TSL to support the roof of a rectangular roadway 5 m wide and 3 m high located 400 m below the ground surface was simulated with FLAC\textsuperscript{3D}. A vertical plane of symmetry through the centre of the roadway was constructed and only half of the roadway was modelled, which helped to reduce the running time. As shown in Figure 6.6, the geometry of the model was selected as 50 m long, 10 m wide and 100 m high. A coordinate system was defined with the origin being at the floor of the roadway, the x-axis being perpendicular to the roadway in the horizontal direction, y-axis being parallel to the roadway development direction and z-axis pointing upwards.

Three 3-dimensional underground excavation models were constructed. The first model simulated excavating the roadway without any reinforcement, the second model simulated excavating the roadway with only rock bolt reinforcement in the roof and the third model simulated excavating the roadway with TSL and roof bolt reinforcement. The models were constrained in the x-direction on x = 0 m and x = 50 m planes, in the y-direction on y = 0 m and y = 10 m planes, and in the z-direction at the bottom of the model. The top plane of each model was located at 340 m depth. The model contained approximately 29000 zones and was loaded as shown schematically in Figure 6.6.
Figure 6.6 Geometry and initial stress state of the model

The roadway was constructed in a 3 m thick coal seam. The immediate roof of the roadway consisted of a 1 m thick claystone layer, a 1 m thick black shale layer, a 1 m thick gray shale layer, a 0.5 m thick siltstone layer and a 0.5 m thick sandstone layer. The floor of the roadway included a 1 m thick gray shale layer, a 0.5 m thick siltstone layer and a 0.5 m thick sandstone layer. The overburden strata of the roof was relatively competent sandstone with a thickness of 53 m in the model, and the strata underlying the floor was 38 m thick sandstone. The geological column of the model is shown in Figure 6.7.

Figure 6.8 illustrate the three dimensional geometry of the FLAC\textsuperscript{3D} model. The TSL and the rock bolts simulated in the model are shown in Figure 6.9.
<table>
<thead>
<tr>
<th>Lithologic pattern</th>
<th>Rock type</th>
<th>Thickness (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sandstone_2</td>
<td>53</td>
</tr>
<tr>
<td></td>
<td>Sandstone_1</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Siltstone</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Gray shale</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Black shale</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Claystone</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Coal</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Gray shale</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Siltstone</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Sandstone_1</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Sandstone_2</td>
<td>38</td>
</tr>
</tbody>
</table>

Figure 6.7 Geological column of the model

Figure 6.8 Geometry of the FLAC$^3$D model showing coal seam and rocks of various properties
6.3.2 Input parameters

The constitutive model employed for all the coal measure rocks was a strain-softening ubiquitous-joint model, except for sandstone_2 which was simulated with a Mohr-Coulomb constitutive model. The strain-softening ubiquitous-joint model is ideal for simulating laminated coal measure rocks. Except for the dip angle and dip direction, the input parameters for these rocks were acquired from Zipf (2007) and are listed in Table 6.2.

The strain-softening behaviour of the rocks also followed the assumptions given in Zipf (2007). The cohesion of the rocks reduced from the peak to 10% of peak over 5 millistrain of post-failure strain, the tensile strength of the rocks decreased to 0 over 1 millistrain of post-failure strain Zipf (2007).
Table 6.2 Input parameters for the rocks (Zipf 2007)

<table>
<thead>
<tr>
<th>Properties</th>
<th>Coal</th>
<th>Claystone</th>
<th>Black shale</th>
<th>Gray shale</th>
<th>Siltstone</th>
<th>Sandstone_1</th>
<th>Sandstone_2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk modulus (GPa)</td>
<td>2.8</td>
<td>2.5</td>
<td>3.3</td>
<td>4.2</td>
<td>4.7</td>
<td>6.7</td>
<td>8</td>
</tr>
<tr>
<td>Shear modulus (GPa)</td>
<td>0.9</td>
<td>1.2</td>
<td>1.5</td>
<td>1.9</td>
<td>2.8</td>
<td>4</td>
<td>4.8</td>
</tr>
<tr>
<td>Density (kg/m$^3$)</td>
<td>2500</td>
<td>2500</td>
<td>2500</td>
<td>2500</td>
<td>2500</td>
<td>2500</td>
<td>2500</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>0.6</td>
<td>1.2</td>
<td>2</td>
<td>3.3</td>
<td>6</td>
<td>10</td>
<td>12</td>
</tr>
<tr>
<td>Joint cohesion (MPa)</td>
<td>0.3</td>
<td>0.5</td>
<td>1</td>
<td>1.9</td>
<td>4.5</td>
<td>7</td>
<td>--</td>
</tr>
<tr>
<td>Tension (MPa)</td>
<td>0.17</td>
<td>0.3</td>
<td>0.6</td>
<td>1</td>
<td>1.9</td>
<td>3.5</td>
<td>4.2</td>
</tr>
<tr>
<td>Joint tension (MPa)</td>
<td>0.08</td>
<td>0.15</td>
<td>0.3</td>
<td>0.6</td>
<td>1.4</td>
<td>2.3</td>
<td>--</td>
</tr>
<tr>
<td>Internal angle of friction (°)</td>
<td>29</td>
<td>22</td>
<td>23</td>
<td>24</td>
<td>26</td>
<td>30</td>
<td>32</td>
</tr>
<tr>
<td>Joint friction angle (°)</td>
<td>25</td>
<td>21</td>
<td>22</td>
<td>23</td>
<td>25</td>
<td>27</td>
<td>--</td>
</tr>
<tr>
<td>Dilation angle (°)</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>--</td>
</tr>
<tr>
<td>Joint Dilation angle (°)</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>--</td>
</tr>
<tr>
<td>Dip angle (°)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>--</td>
</tr>
<tr>
<td>Dip direction (°)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>--</td>
</tr>
</tbody>
</table>

The rock bolts were simulated with the pile structural elements. The rock bolts were 2.4 m in length and 24 mm in diameter. The rock bolts were installed in a grid 1 m apart along both the x-axis and y-axis direction. The load bearing plates were modelled as an elastic material with the Young’s modulus and Poisson’s ratio being 200 GPa and 0.3 respectively. The input parameters for the rock bolts were acquired from Cong (2008) and are listed in Table 6.3.
Table 6.3 Input parameters for rock bolts (Cong 2008)

<table>
<thead>
<tr>
<th>Properties</th>
<th>Rock bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (GPa)</td>
<td>200</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>Perimeter (m)</td>
<td>$7.5 \times 10^{-2}$</td>
</tr>
<tr>
<td>Cross-sectional area (m$^2$)</td>
<td>$4.5 \times 10^{-4}$</td>
</tr>
<tr>
<td>Normal coupling spring cohesion per unit length (N/m)</td>
<td>$1 \times 10^{20}$</td>
</tr>
<tr>
<td>Shear coupling spring cohesion per unit length (N/m)</td>
<td>$1 \times 10^{20}$</td>
</tr>
<tr>
<td>Normal coupling spring friction angle (°)</td>
<td>45</td>
</tr>
<tr>
<td>Shear coupling spring friction angle (°)</td>
<td>45</td>
</tr>
<tr>
<td>Normal coupling spring stiffness per unit length (N/m)</td>
<td>$1 \times 10^{9}$</td>
</tr>
<tr>
<td>Shear coupling spring stiffness per unit length (N/m)</td>
<td>$0.5 \times 10^{9}$</td>
</tr>
<tr>
<td>Second moment with respect to pileSEL y-axis (m$^4$)</td>
<td>$1.6 \times 10^{-8}$</td>
</tr>
<tr>
<td>Second moment with respect to pileSEL z-axis (m$^4$)</td>
<td>$1.6 \times 10^{-8}$</td>
</tr>
<tr>
<td>Polar moment of inertia (m$^4$)</td>
<td>$3.2 \times 10^{-8}$</td>
</tr>
</tbody>
</table>

The 5 mm thick TSL bonded to the roadway roof was modelled with the liner structural elements which are able to not only provide the shear frictional interaction between the liner and the FLAC$^{3D}$ grid but also represent the tensile and compressive behaviour between the two mediums. The input parameters for the TSL are listed in Table 6.4. As mentioned in chapter 3, the Young’s modulus and Poison’s ratio of the TSL were found to be 9.4 GPa and 0.33 respectively. Tests in chapter 3 also demonstrated that the tensile bond strength of the TSL on dry sandstone was greater than 3.1 MPa and the shear bond strength of the TSL on dry sandstone was 10.2 MPa. As such, the normal coupling spring tensile strength and shear coupling spring cohesion of the TSL were set to be 3.1 MPa and 10.2 MPa respectively. The normal coupling spring stiffness and shear coupling spring stiffness of the TSL were calculated using the following equation suggested in the FLAC$^{3D}$ manual:

$$ k_n = k_s = max \left[ \frac{k + 2G}{\Delta z_{min}} \right] \times 10 $$  \hspace{1cm} (6.1)

Where:
\(k_n\) = normal coupling spring stiffness per unit area (N/m\(^3\))

\(k_s\) = shear coupling spring stiffness per unit area (N/m\(^3\))

\(K\) and \(G\) = the bulk and shear modulus respectively (N/m\(^2\))

\(\Delta z_{\text{min}}\) = the smallest dimension of an adjoining zone in the normal direction (m)

Table 6.4 Input parameters for the TSL

<table>
<thead>
<tr>
<th>Parameter</th>
<th>TSL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (GPa)</td>
<td>9.4</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.33</td>
</tr>
<tr>
<td>Thickness (m)</td>
<td>0.005</td>
</tr>
<tr>
<td>Normal coupling spring tensile strength (MPa)</td>
<td>3.1</td>
</tr>
<tr>
<td>Shear coupling spring cohesion (MPa)</td>
<td>10.2</td>
</tr>
<tr>
<td>Normal coupling spring stiffness per unit area (N/m(^3))</td>
<td>(2 \times 10^{11})</td>
</tr>
<tr>
<td>Shear coupling spring stiffness per unit area (N/m(^3))</td>
<td>(2 \times 10^{11})</td>
</tr>
</tbody>
</table>

6.3.3 In situ stress

The 3 m thick coal seam was located 400 m below the ground surface and the top of the model was 340 m deep. The in situ vertical stress applied at the top of the model was calculated to be 8.5 MPa based on an average rock density of 2500 kg/m\(^3\). Assuming the initial horizontal stresses in the x-axis direction (\(\sigma_{xx}\)) and y-axis direction (\(\sigma_{yy}\)) for sandstone_2 were 25 MPa and 15 MPa respectively. These lateral stresses are typical in Australian sedimentary strata as measured in underground coal mines. The initial pre-mining stresses \(\sigma_{xx}\) and \(\sigma_{yy}\) that were applied within the modelled strata and on their boundaries at various depths and rock of various stiffness can be calculated using the following equation (Nemcik 2014):

\[
\sigma_{NL} = \frac{E_N}{E_M} \left( \sigma_{ML} - \frac{\sigma_{xy}}{1-v} \right) + \frac{\sigma_{xy}}{1-v} \tag{6.2}
\]

Where:

\(\sigma_{NL}\) = Normalised horizontal stress (MPa)

\(E_N\) = Normalised Young’s modulus (GPa)
\( E_M = \text{Measured Young’s modulus (GPa)} \)

\( \sigma_v = \text{Vertical stress (MPa)} \)

\( v = \text{Poisson’s ratio} \)

The initial stress states for the rocks in this model are listed in Table 6.5.

<table>
<thead>
<tr>
<th>Rock type</th>
<th>( \sigma_{xx} ) (MPa)</th>
<th>( \sigma_{yy} ) (MPa)</th>
<th>( \sigma_{zz} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal</td>
<td>8.8</td>
<td>6.7</td>
<td>8.5</td>
</tr>
<tr>
<td>Claystone</td>
<td>9</td>
<td>6.5</td>
<td>8.5</td>
</tr>
<tr>
<td>Black shale</td>
<td>10.8</td>
<td>7.4</td>
<td>8.5</td>
</tr>
<tr>
<td>Gray shale</td>
<td>12.5</td>
<td>8.4</td>
<td>8.5</td>
</tr>
<tr>
<td>Siltstone</td>
<td>14.6</td>
<td>8.8</td>
<td>8.5</td>
</tr>
<tr>
<td>Sandstone_1</td>
<td>21.3</td>
<td>13</td>
<td>8.5</td>
</tr>
<tr>
<td>Sandstone_2</td>
<td>25</td>
<td>15</td>
<td>8.5</td>
</tr>
</tbody>
</table>

6.3.4 Results and discussions

6.3.4.1 Tensile bond and shear bond stress at the TSL-rock interface

The TSL together with the substrate forms a strong composite layer if bonded together. Therefore the TSL tensile and shear bond strengths are very important. This is in agreement with many researchers (Archibald 2001; Ozturk 2005 & Yilmaz 2011). Thus the performance of a TSL in roof support is believed to be significantly affected by the TSL’s tensile and shear bond strength. Detachment or slide may occur at the interface between the TSL and rock if the tensile and shear bond strength of the TSL were not strong enough. This would compromise the load transfer ability of the TSL. Figure 6.10 and Figure 6.11 illustrate the maximum tensile and shear stress at the TSL-roof rock interface during the modelling respectively. It is clear from these figures that the maximum tensile bond stress and shear bond stress generated were 0.4 MPa and 1.7 MPa respectively, which were much less than the ultimate tensile bond strength and shear bond strength of the TSL in the model. The results indicated that the TSL did not detach from the rock surface and slide did not occur at the interface.
6.3.4.2 Displacement

In order to investigate the effect of TSL on underground coal mine roof support, three models were executed, one without any roof support, one with only rock bolt support, and one with TSL and rock bolt support. All the models were run until stress equilibrium was reached. Figure 6.12 illustrate the displacements in the $z$ direction of the grid at location (0 m, 0 m, 3 m) at the centre of the roadway with the three support
methods. The model without the support indicated that the grid experienced a vertical displacement of 40 mm after excavating 10 m of the roadway. When excavating 10 m in the roadway supported with rock bolts, the displacement was significantly lower at 25 mm. The displacement was further decreased to only 14 mm when the roof was supported with TSL and rock bolts. This clearly indicated that the TSL is able to control roof sag.

![Graph showing vertical displacements](image)

Figure 6.12 Vertical displacements of the grid (0, 0, 3) m at the roadway crown for the models

Figure 6.13 to Figure 6.15 show the contours of vertical displacements at the centre of the roadway after excavation for the three models. It is apparent that while the maximum vertical displacement of the unsupported roof reached 41 mm, the maximum vertical displacement of the rock bolt supported roof decreased remarkably to 31 mm and the maximum vertical displacement further reduced to only 14 mm when the roof was supported with TSL and rock bolts. In addition, the floor heave of the unsupported roadway was also greater than that of the rock bolt model and TSL-rock bolt model, being 18 mm, 15 mm and 14 mm respectively. The results demonstrated that TSL-rock bolt roof support can not only stabilize the roof but also help to control floor heave.
Figure 6.13 Contour of vertical displacements at the centre of the roadway after excavation for the model without support

Figure 6.14 Contour of vertical displacements at the centre of the roadway after excavation for the model with rock bolt support
The contours of x-displacement of the three models after roadway excavation are illustrated in Figure 6.16 to Figure 6.18. It is interesting to note that in the unsupported rib the maximum rib displacement with the rock bolt roof support model was 46 mm, which was even 1 mm greater than that of the roof unsupported model. The TSL supported roof model had the smallest maximum rib displacement of 41 mm. These results indicated the roof TSL support helps to alleviate buckling of the unsupported coal rib, which is beneficial to roadway stability.
Figure 6.17 Contour of x-displacements after roadway excavation for the model with rock bolt support

Figure 6.18 Contour of x-displacements after roadway excavation for the model with TSL and rock bolt support

Figure 6.19 compares the roof displacements of all three models. It is apparent that while the deformation zone reached 5 m into the unsupported roof the deformation zone in both the rock bolt supported and the TSL-rock bolt supported roof was 3.5 m,
which clearly shows the support effectiveness of the TSL and rock bolts. In the roof zone between 2 to 3 m above the roof, the deformation characteristics of the unsupported model and the TSL-rock bolt supported model were similar, and the rock bolt supported model had the greatest displacement. In the first 2 m above the roof level, it was relatively more difficult for the TSL and rock bolt supported roof to deform as a result of the constraint provided by the TSL-rock bolt support. Reducing the height of the deformation zone is beneficial to roof stability as it helps to keep the integrity of the rock mass and thus preserve its inherent strength.

![Graph showing roof displacement](image)

**Figure 6.19** Roof displacements of all three models

Figure 6.20 illustrates the unsupported rib displacements of the three models. Unlike the roof, the effect of the TSL-rock bolt roof support was not so significant for the rib displacements. The rib deformation characteristics of the three models, as shown in Figure 6.20, were similar. For the unsupported roadway model displacements greater than 1 mm extended 18 m into the rib, and 16 m into the rib for the TSL-rock bolt supported model. It is important to note that the deformation zone reached 22 m into the rib for the rock bolt supported model. The maximum rib displacements for the three models were similar, ranging from 40 mm to 45 mm.
6.3.4.3 Yield zones

The yield zones of the unsupported and TSL-rock bolt support models shown in Figure 6.21 and Figure 6.23 indicated that the TSL-rock bolt support did not have much effect on the yield zone of the roadway. In both models, the yield zone extended 2 m into the roof and 1 m into the floor. Note that the yield zone patterns and displacements within the unsupported rib were very similar in both cases but marginally larger in case of the unsupported roadway. Specifically, the extension of the yield zone in the rib of the unsupported roadway model was approximately 7.8 m while the yield zone in the supported model reached around 9.8 m into the rib. The yield zone pattern of the rock bolt supported model, shown in Figure 6.22, was slightly different from that of the other two models. The extension of the yield zone into the roof in the rock bolt supported model was 3.3 m, which was greater than that of the other two models, however, the extension of the yield zone in the rib in this model was the smallest among the three models, being 7.3 m. The yield zone in the floor of the rock bolt support model was identical to that of the other two models. These yield zones appeared more extensive than would be normally measured underground using an extensometer, however the instrumentation is usually installed at the already excavated roadway coal face thus the initial displacements that would normally indicate early yielding cannot be recorded.
Figure 6.21 Yield zone of the model without roadway support

Figure 6.22 Yield zone of model with rock bolt roof support
Figure 6.23 Yield zone of the model with TSL-rockbolt roof support

6.3.4.4 Effect of mining depth on the TSL roof support performance

In order to investigate the sensitivity of mining depth on the roof support performance of the TSL, a set of numerical models were constructed. The mining depths studied are listed in Table 6.6. As the mining depth altered, the corresponding initial vertical stress also varied. The vertical stress was calculated using equation (2.1) and is presented in Table 6.6. Note that as 60 m thick overburden strata was constructed in the model, its weight was excluded from the initial vertical stress. The ratios of the principal and minor horizontal stresses versus vertical stress were kept consistent, being 2.5 and 1.5 respectively.

Table 6.6 Mining depths and corresponding vertical stresses

<table>
<thead>
<tr>
<th>Mining depth (m)</th>
<th>200</th>
<th>300</th>
<th>400</th>
<th>500</th>
<th>600</th>
<th>700</th>
<th>800</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial vertical stress (MPa)</td>
<td>3.5</td>
<td>6</td>
<td>8.5</td>
<td>11</td>
<td>13.5</td>
<td>16</td>
<td>18.5</td>
</tr>
</tbody>
</table>

Figure 6.24 compares the maximum roof displacements of the three models as a function of various mining depths. It is clear from the results that roof displacements in the three models experienced similar trends. Roof displacements increased slightly from a depth of 200 m to 300 m in all the models, and then underwent a significant
increase at a depth of 400 m. When mining at depths greater than 400 m, the increase in roof displacement was not so significant. The results also indicated that the effect of the TSL on controlling roof deformation was more significant under deep mining depth. At shallow depth of 200 m, roof displacements of the three models were close. The difference in roof displacement among the three models increased slightly when mining depth increased to 300 m, with the roof displacement of unsupported roadway model being 13 mm, the rock bolt supported model being 10 mm and TSL-rock bolt support being 7 mm. The difference kept growing as mining depth increased. Specifically, when mining depth increased from 400 m to 800 m, the displacement of the unsupported roof increased from 41 mm to 61 mm, the displacement of the rock bolt supported roof decreased from 31 mm to 27 mm at first and then grew to 34 mm, the displacement of the TSL-rock bolt supported roof experienced a slight increase, from 14 mm to 18 mm. This indicated that the effect of the TSL-rock bolt support controlling roof deformation was more significant for greater mining depth.

Figure 6.24 Maximum roof displacements of the two models as a function of various mining depths

The maximum rib displacements of the three models at various mining depths are shown in Figure 6.25. The results indicated that rock bolt roof support and TSL-rockbolt roof support had almost identical influence on rib displacement. Both the support methods had negligible effect on the rib displacement when the mining depth was less than 500 m, however, a significant rib displacement of 13 mm between the
unsupported roadway and roof supported roadway models occurred when the mining depth was greater than 500 m.

![Figure 6.25 Maximum rib displacements of the two models as a function of various mining depths](image)

6.3.4.5 Effect of the ratio of principal horizontal and vertical stress on the roof support performance of the TSL

Another set of numerical models were constructed to study the effect of different ratios of principal horizontal stress ($\sigma_{xx}$) versus vertical stress on the roof support behaviour of the TSL. The ratios selected were $1.5 : 1$, $2 : 1$, $2.5 : 1$ and $3 : 1$. The minor horizontal stress ($\sigma_{yy}$) was kept constant at 15 MPa for all the models. Note that the input initial vertical stress was 8.5 MPa instead of 10 MPa as explained previously. The input parameters are listed in Table 6.7.

The maximum roof displacements of the three models under the condition of various horizontal versus vertical stress ratios are shown in Figure 6.26. It is obvious that the TSL-rock bolt roof supported model had the best performance in controlling roof deformation. Specifically, the maximum roof displacement of TSL-rock bolt roof support model fluctuated between 14 and 19 mm at the various horizontal versus vertical stress ratios. The rock bolt supported model was greater ranging from 19 mm to 36 mm. As expected, the unsupported roof experienced the greatest roof
displacement ranging from 35 to 54 mm for the same modelled cases. It is important to note that except for the ratio of 2.5 : 1 where the rock bolt roof support model had identical maximum roof displacement to the TSL-rock bolt supported model, the maximum roof displacement of the bolt only support was always greater than with the TSL-rock bolt in all the other cases with the difference of approximate 17 mm. This clearly indicated that the TSL was able to control the roof deformation, helping to maintain the roadway stability.

Table 6.7 Input parameters for different principal horizontal stress versus vertical stress ratios

<table>
<thead>
<tr>
<th>Ratio of principal horizontal stress versus vertical stress</th>
<th>$\sigma_{xx}$ (MPa)</th>
<th>$\sigma_{yy}$ (MPa)</th>
<th>$\sigma_z$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5 : 1</td>
<td>15</td>
<td>15</td>
<td>8.5</td>
</tr>
<tr>
<td>2 : 1</td>
<td>20</td>
<td>15</td>
<td>8.5</td>
</tr>
<tr>
<td>2.5 : 1</td>
<td>25</td>
<td>15</td>
<td>8.5</td>
</tr>
<tr>
<td>3 : 1</td>
<td>30</td>
<td>15</td>
<td>8.5</td>
</tr>
</tbody>
</table>

Figure 6.26 Maximum roof displacements of the two models as a function of various ratios of principal horizontal stress versus vertical stress
Figure 6.27 illustrates the effect of different horizontal versus vertical stress ratios on the rib displacement of the three models. It is apparent that neither the TSL-rock bolt roof support nor the rock bolt roof support affected the rib displacement at the given horizontal versus vertical stress ratios. The maximum rib displacements of all the three models were quite close when subjected to the four different ratios.

Figure 6.27 Maximum rib displacements of the two models as a function of various ratios of principal horizontal stress versus vertical stress

6.4 Detailed modelling of roof support performance of thin spray-on liners

The modelling results indicate that most of the strata reinforcement work is done by bolts. Bolts provide a normal stress to the fractures and thus minimise their displacements when additional stresses are applied. The fully encapsulated bolts clamp fractured strata most effectively adjacent to the bolt, however their clamping effectiveness reduces with distance from the bolt (SCT n.d.2). The actively reinforced strata zones adjacent to the bolts are depicted in Figure 6.28. There are zones between the bolts at lower roof horizon that cannot be reinforced by bolts alone. If these zones are severely broken, they can fall out and compromise efficiency of the bolting system. Therefore the role of TSL support is not to provide an overall strata support but to offer an effective support to the substrate skin in the area between the bolts.
The TSL together with the substrate skin form a composite layer that is superior to the passive steel mesh due to its reinforcing capabilities. The usual volume of rock that needs to be supported by the TSL is approximately a pyramid in shape with the base defined by the area between the four adjacent bolts and the height depending on the bolt spacing and their reinforcing effectiveness. As the bonding characteristics of the TSL material have proven to provide effective reinforcement to fractured rock/coal skin, the system minimises the possibility of roof falls between the bolts and thus assist in the overall support efficiency to preserve the confining stress within yield/fractured strata and thus minimise strata displacements. Three-dimensional modelling work was undertaken to quantify the above discussed issues. This involved a detailed model of the bolt-unreinforced zones at the roof level supported by TSL, as shown in Figure 6.29.
6.4.1 Description of the model

Two models were created, while one model was reinforced with rock bolts only, the other was reinforced with TSL and rock bolts. Figure 6.30 and Figure 6.31 illustrate the geometry and rock geology of the model and the arrangement of rock bolts respectively.

Figure 6.30 Geometry and strata geology of the rock bolt supported model

Figure 6.31 Arrangement of rock bolts in the model
The model geometry was 5 m wide across the mine roadway (x-axis), 3.2 m long (y-axis) and 3 m high (z-axis), with the x = 0 plane located in the middle of the model, the y = 0 plane located at the front surface of model and the z = 0 plane located at the roof surface. The roof strata in the model consisted of one layer of 0.2 m thick claystone, two layers of 0.2 m thick black shale, two layers of 0.2 m thick gray shale, one layer of 0.4 m thick siltstone, one layer of 0.4 m black shale, one layer of 0.6 m thick siltstone and one layer of 0.6 m thick sandstone. All of the rocks were modelled with strain-softening ubiquitous-joint constitutive model and the rock properties were the same as mentioned in section 6.3 above. Rock bolts in both models were modelled with pile structural elements and they were set 1 m apart in the centre of the model as shown in Figure 6.31. A 5 mm thick TSL liner, which was modelled with liner structural element, was bonded on the roof surface. The properties of the TSL, rock bolts and load bearing plates were also the same as those in section 6.3.

Both roadway roof models were fixed in the vertical direction at the modelled top surface of the strata and at both sides. Horizontal velocities of $1 \times 10^{-5}$ m/step and $-1 \times 10^{-5}$ m/step were applied on the left and right side surfaces of the model respectively.

6.4.2 Results and discussions

6.4.2.1 Displacements in the modelled strata

In order to study the influence of the TSL on the roof support performance, both models were compressed 27 mm from each side in the horizontal direction. The compression was to simulate underground lateral stresses within the roof strata and achieve reasonable strata yielding and vertical roof displacement as typically measured underground. This loaded roof state allowed comparison of the roof displacements for various support systems. The vertical displacement contours shown on a 1 m$^2$ cross-section at the roadway roof centre are shown in Figure 6.32 for rock bolts only and in Figure 6.33 for TSL-rock bolts. As expected, the maximum vertical displacement of roof strata in the rock bolt supported model was greater than that in
the TSL-rock bolt supported model. The maximum displacement in the non-TSL case was approximately 78 mm which was 20 mm higher than with the TSL reinforcement.

Figure 6.32 Vertical displacement contour (1 m$^2$ of the cross-section at the roadway roof centre) in the rock bolt supported model

Figure 6.33 Vertical displacement contour (1 m$^2$ of the cross-section at the roadway roof centre) in the TSL-rock bolt supported model

The vertical displacements at the roof centre point in both models were compared in Figure 6.34. It is apparent that the vertical displacement in the TSL-rock bolt reinforced model was always smaller than that in the rock bolt supported model. For example, while the roof was compressed by 6 mm at the model sides the vertical
displacement at the roof level in the rock bolted strata reached 6.2 mm and the corresponding vertical displacement of the TSL-rock bolt model was only 0.8 mm. This clearly indicated that the TSL was effective in preventing the rock skin from deforming. Minimising the deformation of rocks is beneficial to roof stability as it contributes to preserving a small percentage of its original strength. The results also indicated that the displacement difference between the two models increased slightly as the compression displacement increased.

Figure 6.34 Vertical displacements at the roadway central point at the roof surface (both models)

6.4.2.2 Stresses within the modelled roof strata

Figure 6.35 and Figure 6.36 show the horizontal stress in the x-axis direction in the central bottom part within a 1 m³ block of the rock mass at the roadway roof centre of the rock bolt supported model and TSL-rock bolt supported model respectively. It is apparent that the horizontal stress in the case of rock bolt only model was smaller than that in the TSL-rock bolt supported model. In the case of rock bolt only model, the horizontal stress was just above 0 MPa within the range from the bottom surface up to 0.2 m into the roof. The horizontal stress increased approximately from 0.5 MPa to 1.5 MPa from 0.2 to 0.4m above the roof. The maximum horizontal stress at the bottom surface in the TSL-rock bolt case was significantly higher reaching in some places up to approximately 6 MPa (the stress at the load bearing plate was excluded). The stress from 0.2 m to 0.4 m in the rock mass was approximately 5 MPa. The stress higher up into the roof increased at a higher rate compared to the rock bolt case. These results
indicate that the TSL was able to help reinforce the rock mass and provided significant confinement to the strata, which made the strata stronger.

Figure 6.35 Horizontal stress cross-section within the 1 m³ block of the rock mass at the roadway roof centre for rock bolts only

Figure 6.36 Horizontal stress cross-section within the 1 m³ block of the rock mass at the roadway roof centre for TSL-rock bolt model

Similar results were observed when comparing the vertical stresses in the central part of the mine roadway, rock bolt only (Figure 6.37) and TSL-rock bolt supported model (Figure 6.38). The vertical stress at the bottom surface of the rock bolt supported model was nearly zero (stress at the load bearing plate was excluded). The maximum vertical stress at the roof surface of the TSL reinforced model was approximately 5
MPa. Results also showed that the vertical stress in the rock up to 1 m into the roof in the TSL supported model was much greater than that in the rock bolt supported model.

![FLAC3D 5.00](image1)

Figure 6.37 Vertical stress cross-section within the 1 m$^3$ block of the rock mass at the roadway roof centre for rock bolts only

![FLAC3D 5.00](image2)

Figure 6.38 Vertical stress cross-section within the 1 m$^3$ block of the rock mass at the roadway roof centre for TSL-rock bolt model

The modelling results indicated that, TSL bonding to the rock surface formed a composite layer with the substrate skin, which enhanced the rock strength and ultimately improved the roof stability.
6.5 Summary and conclusions

In this chapter, the three-point bending test of steel strands was modelled. Additional numerical models were developed to simulate the full scale pull test on steel mesh. The results from the numerical modelling matched well with the results from the laboratory tests. The effects of flat plate loading and dome loading were also compared using numerical modelling. These tests indicated that the two loading types produced similar results except that flat plate loading produced slightly higher load at first wire failure and stiffer initial load-displacement behaviour, which was due to the fact that in the initial period the applied load was carried by more wires in the flat plate loading test and the load appeared to be transferred to the bolt more effectively.

The performance of a TSL-rock bolt system used in underground coal mine roof support was also studied using numerical modelling. It was shown that the TSL-rock bolt system was able to significantly reduce the roof displacements and sustain higher stresses within the supported roof and thus help to maintain roof stability. The results also indicated that TSL-rock bolt roof support did not have significant effect on decreasing rib displacements and yield zones. However, it was able to reduce the extension of the deformed rock zone in both the roof and rib, which helps to preserve the integrity of rock mass. The effects of mining depth and horizontal stress versus vertical stress ratio on the rock support performance of the TSL were also studied using numerical modelling. It was shown that the TSL-rock bolt was effective in preventing roof deformation even at a mining depth of 800 m and a high horizontal to vertical stress ratio of 3 : 1.

Detailed models were also made to study the TSL roof support behaviour within the severely yielded roof strata. The modelling results indicated that with TSL reinforcement, the roof displacement decreased and both horizontal and vertical stresses in the roof were higher than in the bolt supported roofs without TSL, thus increasing the roof rock mass strength.
7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions
The research presented in this thesis was used to support the development of thin spray-on liners for use in supporting the roof and ribs of an underground coal mine development heading, the following conclusions can be drawn:

The strength of hydrostone plaster, frequently used as rock simulation material in the laboratory, was affected by the curing time and environment. As previous studies on this topic were limited, a series of plaster cylinders and beams with different curing times and environments were tested to investigate the influence the two factors had on the compressive and flexural strength of the plaster. As hydrostone plaster was used as the substrate for the TSL-plaster composite in this thesis, the test results helped to determine the curing time and environment so that the plaster was in the stable stage.

The TSL mechanical properties, such as tensile strength and adhesion strength, have been tested by many researchers in the past, however, the Poisson’s ratio of a TSL has rarely been investigated. As Poisson’s ratio is a very important input parameter in the numerical modelling, it was studied in this thesis. Tensile strength, Young’s modulus, flexural strength, flexural modulus and Poisson’s ratio of polymer A, B, C and their fibre reinforced versions were measured. Polymer adhesion tests were conducted following the test method developed by Tanant and Ozturk (2004). The adhesion strength of polymer A and B on dry and wet sandstone could not be accurately determined as the sandstone was so weak that all of the failure occurred in the sandstone, which indicated that the adhesion strengths of both polymers were greater than the tensile strength of the sandstone (3 MPa). The double-sided shear strength (DSS) test proposed by Saydam et al. (2003) was modified slightly by adding four more bolts to the test rig in order to eliminate the influence of bending. The improved test method was used to evaluate the shear bond strength of polymers A and B on dry and wet sandstone. The results showed that moisture had a negative influence on the
polymer shear bond strength, thus, it is recommended that the excavation surface be as dry as practicable before applying the TSL.

Tensile, bending and weld shear tests were conducted on the steel strands that are fabricated to form the steel mesh currently used in Australian coal mines, the test results provided input parameter for the numerical modelling conducted in this study. None of the steel wires experienced weld shear failure during the weld shear test, so this failure type was not considered in the numerical modelling.

Beam enhancement is one of the support mechanisms of TSLs, however, investigation on this mechanism is limited. This study investigated the beam enhancement capability of the TSL by the use of four-point bending test on plaster beams with and without TSL reinforcement, providing a better understanding of this support mechanism. The results indicated that a 5 mm thick TSL significantly improved the strength of the plaster beam, with the peak load of a reinforced plaster beam being more than double that of the plain plaster beam. The TSL reinforcement also altered the failure mode of the samples. While the plain plaster beam failed in tension the TSL-plaster composite experienced shear failure. More importantly, even though plaster beams fail in a brittle way, beams reinforced with a TSL showed various levels of residual strength of 2 kN to nearly 5 kN. The residual strength of a TSL-rock composite is desirable for underground coal mine roof support as it is still able to provide resistance after failure, which eliminates sudden fall of roof rocks and ultimately reduces the potential injury for mining personnel.

As a TSL is able to penetrate into the cracks in the roof, an ability which steel mesh does not have, a series of innovative laboratory tests were designed to investigate the mechanism by which TSLs help to stabilise the cracked roof or rib of an underground roadway, helping to evaluate the feasibility of replacing steel mesh with the TSL in roof support. The crack in the rock mass was modelled using a rectangular or v-shape notch in plaster beams which were subjected to a four-point bending test. Test results
indicated that even if the crack wasn’t filled with TSL, the TSL was able to significantly increase the strength of the cracked rock. Further improvement in strength was achieved once the TSL penetrated into the rock cracks. Results in all cases also showed that failure at the crack tip for the unreinforced beams had been resisted and that failure of the beam initiated elsewhere. This indicated that the TSL-rock composite was also able to reduce the stress concentration at the crack tip and thus resist crack propagation in the rock mass, an ability which conventional steel mesh does not have. Additionally, test results demonstrated that reinforced plaster beams with the notch filled with polymer experienced plastic failure instead of brittle failure, indicating the penetration of TSL material into the cracked roof would change its failure mode. Compared with brittle failure, plastic failure of the rock support medium is preferred in underground coal mines as it is able to provide a warning to mining personnel, helping to improve mining safety.

Previous tests on TSLs were mainly on determining their basic mechanical properties. This study, for the first time, designed many test methods to investigate the behaviour of TSL in supporting strata with weak bedding planes, buckling strata and strata prone to guttering. The strata with weak bedding planes was modelled with concrete block embedded with several thin plastic plates, the buckling strata was simulated with curved plaster slabs and strata prone to guttering was modelled using many concrete prisms which were layered and placed in a steel frame. Test results showed that the application of the TSL reinforcement helped to improve the strength of the modelled strata in all the cases, demonstrating the potential of the TSL for rock support in underground coal mines.

TSLs have been stated as a potential alternative to steel mesh for underground coal mine roof support for a long time, however, research on a direct comparison between TSLs and steel mesh in rock support has been limited. As a result, comparison between a TSL and steel mesh in supporting strata with weak bedding planes, buckling strata and strata prone to guttering were compared using the laboratory tests mentioned above.
It was found that the TSL reinforced strata with week bedding planes had a stiffer load-displacement behaviour than the steel mesh confined sample, this was a result of the TSL being bonded to the concrete block and acting as a composite with the concrete at the very beginning of the loading. This indicates that TSLs are superior to steel mesh in restricting the softening of fractured strata. As with the steel mesh confined sample, the TSL reinforced sample showed a similar level of residual strength.

With respect to the buckling test, as expected, both the steel mesh and the TSL helped to significantly increase the strength of the sample. Compared with steel mesh the improvement in strength attributed to TSL reinforcement was even greater, indicating that TSLs were able to provide superior performance than steel mesh in supporting buckling strata. Since the TSL was bonded to the sample and formed a TSL-plaster slab composite, the crack initiating load of the TSL reinforced samples were also greater than the steel mesh confined samples, which demonstrated that the TSLs were better than steel mesh in preventing rock from cracking. In addition to the strength and crack initiating load, the TSLs reinforced samples also had stiffer load-displacement behaviour than the steel mesh confined samples. This was due to the fact that the TSLs were in intimate contacted with the plaster and as such generated resistance at very small deformation. A stiffer surface control system is beneficial to controlling buckling strata in underground coal mines as it is able to produce greater resistance for the same deformation. It was also observed that the sample reinforced with a TSL had a greater peak load, increased effective stiffness and greater residual strength.

With the guttering test, the tests terminated before either the steel mesh or the 5 mm thick TSL failed due to safety concerns. As such, the peak loads of the steel mesh and TSL reinforced strata prone to guttering could not be compared. However, test results indicated that in contrast to steel mesh, the TSL showed significantly stiffer support performance, which was desirable for guttering roof support as it enabled the fractured rock to maintain its self-supporting ability and as a result helped to prevent guttering from developing.
The majority of the previous laboratory tests on determining the load bearing capacity of TSLs and steel mesh were relatively small scale, which may not accurately represent the in situ behaviour of the surface control mediums. Therefore, full scale laboratory tests were conducted in this study to compare the load bearing capacity of steel mesh and TSLs. Additionally, in order to simulate the in situ situation in underground coal mines, one of the TSL sheets was bonded with cracked concrete blocks and subjected to the testing. Test results demonstrated that even though the plain fibre reinforced polymer liner sheets were not as strong as the roof mesh, they were stronger than the rib mesh. When the TSL was bonded with concrete blocks and formed a TSL-concrete composite, its load bearing capacity improved significantly and was as strong as the roof mesh. It was also observed that both the plain TSL sheet and TSL-concrete composite were much stiffer than the roof mesh. It is not practical to prevent the formation of mining induced fractures but it is possible to enhance the excavation surface condition by applying an effective support system at an early stage of mining. The nature of rock support is to preserve the rocks self-supporting ability by the use of the rock support material rather than holding the dead weight of the rock. As stated previously, the stiff behaviour of TSLs is able to reduce rock mass dilation or deformation, helping preserve the inherent strength of the rock mass and maintaining rock stability.

The load bearing capacities of the TSLs and traditional steel mesh were compared using full scale laboratory tests for the first time. Moreover, the behaviour of the TSL in supporting rock with weak bedding planes, buckling strata and strata prone to guttering was also compared to that of steel mesh. The comparisons between the TSL and steel mesh in this regards were made for the first time, which helped to analyse the feasibility of using TSL as an alternative to steel mesh in roof support.

In addition to laboratory tests, numerical modelling was also performed in this study. The full scale laboratory test on steel mesh was successfully modelled with numerical simulation. It is important to note that the modelling was only conducted to the point where the first wire broke, as input parameters such as slippage were not available.
Modelling indicated that a domed loading platen and flat plate loading platen would produce similar results in the full scale laboratory test on steel mesh. The initial load-displacement behaviour in the flat plate loading was slightly stiffer. This occurred because in the initial period the load was carried by more wires and the load was more effectively transferred to the bolts in this case.

Three-dimensional numerical models were also developed to study the behaviour of a TSL in supporting the roof of underground coal mine roadways for the first time. It was confirmed that the application of the TSL together with rock bolts was able to significantly reduce roof displacements by up-to 55% compared with rock bolt only support. The range of the rock deformation zone in both the roof and rib decreased, which was beneficial to roadway stability, as rock deformation was minimised and rock mass inherent strength was preserved. In addition, numerical models were also constructed to investigate the influence of mining depth and horizontal to vertical stress ratio on the roof support performance of TSL-rock bolt system. Results indicated that the TSL-rock bolt was effective in minimising roof deformation even at a mining depth of 800 m and a high horizontal to vertical stress ratio of 3 : 1.

Detailed models were also constructed to study the TSL roof support behaviour within the severely yielded roof strata. It was observed that with TSL reinforcement, the roof displacement decreased and both horizontal and vertical stresses in the roof were higher than in the roof supported only by bolts, thus showing an increase in the roof rock mass strength with TSL reinforcement.

In final conclusion, the results from the studies in this thesis have shown that a TSL has the potential to replace steel mesh as a component of the primary support in development headings in underground coal mines. The application of a TSL can be automated, as can roof bolting, this has the potential to significantly increase development rates as well as removing mine personnel from the potentially dangerous situation of working under unsupported roof.
7.2 Recommendations for future research

The following recommendations are made for future research:

(i) Numerical modelling of steel mesh in underground coal mine roof support needs to be conducted to compare with the TSL.

(ii) In situ application of the TSL needs to be conducted to verify the results from the numerical modelling.

(iii) Conducting sensitivity studies of various cases involving TSL thickness, interface bonding characteristics, rock strength, stress levels, rock bolt properties and their spacing etc. that may affect the support performance of a TSL.

(iv) Cost of applying TSL for underground coal mine roof support needs to be studied, this cost should include not only the expenditure on the material but also the time needed for support installation. Moreover, the cost of TSL application should be compared with that of mesh application, thus the feasibility of replacing steel mesh with TSL can be made in terms of economic aspects.
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