Development of a laboratory facility for testing shear performance of installed rock reinforcement elements

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DEVELOPMENT OF A LABORATORY FACILITY FOR TESTING SHEAR PERFORMANCE OF INSTALLED ROCK REINFORCEMENT ELEMENTS

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2006

A thesis submitted in partial fulfilment of the requirements for the award of Master of Engineering (Mining)
The University of New South Wales
I hereby declare that this submission is my own work and to the best of my knowledge it contains no materials previously published or written by another person, nor material which to a substantial extent has been accepted for the award of any other degree or diploma at UNSW or any other educational institution, except where due acknowledgment is made in the thesis. Any contribution to the research by others, with whom I have worked at UNSW or elsewhere, is explicitly acknowledged in the thesis.

I also declare that the intellectual content of this thesis is the product of my own work, except to the extent that assistance from others in the project's design and conception or in style, presentation and linguistic expression is acknowledged.

Signed on this ____________________ day of ____________________ 2006

_________________________________
ABSTRACT

Rock reinforcing elements provide a significant proportion of their ground control capability through offering resistance to shear movement of adjacent rock masses or blocks. This potential shear movement may take the form of sliding on horizontal bedding planes leading to strata bending; or block displacement along other geological structures such as joints or similar discontinuities.

Much has been reported about this type of behaviour of rock bolts and other tendons, in theoretical concepts. However, there is a shortage of quality data available on the exact nature of this mechanism for shear resistance, and the role played by parameters such as pre-tensioning. A clearer understanding of the nature and significance of this type of behaviour has major implications for rock reinforcing materials and installation design.

This thesis, which was supported by the Australian Coal Research Program (ACARP) describes the design, construction and commissioning of a laboratory testing facility at the School of Mining Engineering, University of New South Wales (UNSW), Australia and a subsequent testing program. The single failure plane design adopted in the test rig has been successful in allowing shear loading to be directly applied to fully installed rockbolts.

Rockbolts were installed into an offset concrete rockmass, which consisted of two separate concrete samples that created a smooth shear plane surface. The reinforced samples were subjected to an applied shear load and critical parameters such as load and shear displacement were recorded. Influencing parameters such as concrete strength and applied pre-tension were altered and recorded to determine their effects on the overall shear performance of the sample. The failure mode of the rockbolts was also examined.

The results indicate that a relative stronger rockmass material caused the rockbolt to fail within a lower shear displacement compared to a relatively
weaker material. Also, a pre-tensioned rockbolt tended to resist shear displacement at least initially, until high shear loads developed. This phenomena is beneficial to ground support as less movement would tend to maintain integral strength of the rockmass.

The use of strain-gauged rockbolts indicated as would be expected that the shear loading arrangement induced a compressive axial loading that tended to dissipate with distance from the shear surface.
EXECUTIVE SUMMARY

The initial results and conclusions of this research project have been possible with the creation of a laboratory-based shear testing facility within the School of Mining Engineering at the University of New South Wales. This facility has been designed, constructed and commissioned to generate a greater understanding of rock reinforcing elements subjected to shear loading. The project looked at some of the parameters that can influence the ability of a reinforced sample to resist shear movement through the use of this shear testing facility.

Background

This thesis was part of the Australian Coal Association Research Program (ACARP) project C12010 ‘Mechanical Behaviour of Reinforcement Elements Subjected to Shear’. This project was a laboratory-based project to investigate the behaviour of various reinforcement tendons in relation to their role in resisting shear deformation, thereby reinforcing blocky or bedded rock masses. The specific objectives of the project were to:

- Undertake a literature review on shear behaviour of reinforcement elements
- Design, construct and commission a laboratory test facility
- Conduct a laboratory program of experiments evaluating parameters such as the effective shear capacity of a support element, the influence of the loading rate on test outcomes, test variations due to encasing material properties, the contribution of element pre-tensioning to shear capacity and the failure mode of the reinforcing element.
- Analyse the above results, in comparison with theoretical and other models.
- Prepare industry guidelines for application of tendon supports in discontinuous materials, in terms of shear behaviour.

A literature review was undertaken of previous research results related to the performance of bolted rock joints under shear loading. This literature review was initially undertaken by Mr Wouter Hartman and further developed within this thesis. A listing of the parameters that influence the performance of reinforced
rockmasses subjected to a shear load was finalised and a paper was published on the findings of the research within the last 30 years¹;

The findings from the literature study indicated a general agreement among researchers and practitioners regarding the influence of some of the parameters involved but also indicated a minimal understanding of the effect of actual reinforcement installation method and associated loading mechanisms may have on failure. Especially of interest are the effectiveness of pre-tension, torque, axial tension and/or normal loading.

The main focus of the work presented in this thesis was to design, construct and commission a shear testing facility that could be used to analyse the parameters influencing shear loading of reinforced rockmasses. A series of experiments was undertaken and also reported.

**Shear Testing Facility**

The laboratory testing facility was designed to be suitable for full-scale testing of shear performance making use of a hydraulic activated Avery-Denison compressive machine that is located in the School of Mining Engineering, UNSW. The design of this facility best represents and creates the issues that are common in the underground mining environment with the single failure plane design adopted in the test rig successfully allowing shear loading up to 600 kN to be directly applied to fully installed (and where relevant, pre-tensioned) reinforcing elements. The facility has the capability to isolate numerous variables and quantify/analyse these parameters accordingly.

The initial design was based on a single failure plane where a load can be applied to a section of the reinforced specimen on one side of the shear plane

and kept rigid on the alternative side of the shear plane. The reinforcing element was then subjected to a shear load through the shear displacement of the surrounding rockmass relative to one another.

Modifications were made after commissioning during experimentation phase. Monitoring instrumentation incorporated included pressure transducers, digital output panel, displacement transducers (LVDTs), hydraulic load cell and a Data Acquisition System (DAQ) to monitor these parameters during the testing process. The shear testing facility also was modified to allow for the provision of strain-gauged rockbolts.

A test sample was comprised of two concrete blocks cast in customised steel casings. After curing these were temporarily bolted together, offset from one another and a borehole then drilled through the two blocks after which a rockbolt was installed as per current mining practices. Casting the two blocks separately enabled a smooth shear plane to be created to allow the two blocks to slide past one another once a shear load was applied.

Three series of tests were undertaken. Series or Stage 1 included two test samples that were used primarily to determine the functionality of the testing facility and provide initial results and scope for further modifications/adjustments to the facility. Stage 2 saw the introduction of additional instrumentation and modifications to the testing facility. This involved six test samples. Stage 3 saw six test samples and included three strain-gauged rockbolts.

Installation of the reinforcing elements was undertaken by an ARO roofbolter, which is commonly used in underground coal mines throughout the world. Using a roofbolter to install the reinforcing elements raised minor issues in the repeatability of installation practices with variations in the borehole size and level of applied pre-tension.
Results of Investigations

The design and construction of the shear testing facility enabled shear testing of installed reinforcing elements. Numerous modifications and improvements were made throughout the project to achieve a testing facility that could isolate and analyse the parameters that can influence the shear performance of a reinforced rockmass.

Several investigations were conducted while commissioning the shear testing facility including:

- Effect of rockmass material strength
- Difference in loading characteristics between a pre-tensioned and non-tensioned reinforcing element
- Failure modes of the reinforcing element

The various findings are as follows.

- Standard BX rockbolts installed in the concrete rockmass was found to provide a shear resistance higher than the ultimate tensile strength (UTS) of the rockbolt element. There were two distinct loading zones observed in the graphs of the applied shear load versus shear displacement curve. Initially the system was stiff with little displacement observed with continued loading until a point was reached when stiffness was reduced and maintained until failure of the rockbolt. Even after loading was stopped and the shear load re-applied, the subsequent load-displacement curve follows this same loading characteristic.

- Over the range of loading rates examined, at higher loading rates the stiffness of the shear load-displacement curves was much greater than that observed when a much lower loading rate was used; that is, stiffness of the system was a function of the rate of load application.
• Failure of the rockbolt tended to occur after a limited shear displacement in a higher strength rockmass compared to that observed in softer rock materials. This was due to the reduced rockbolt length “activation zone” in the stronger rockmass with minimal crushing around the extent of the borehole in a stronger rockmass material compared to the weaker rockmass.

• When no face plate and nut were used at the collar of the borehole, there was failure of the rock/resin/rockbolt interface as the rockbolt was slowly extruded through the borehole at a constant applied load.

• A pre-tensioned reinforcing element prevented early shear displacement at higher applied shear loads, which was beneficial in minimising initial shear movement within the surrounding rockmass. Beyond initial loading regime, the pre-tensioned element then reduced in stiffness to a consistent level until failure occurred.

• As the level of pre-tension increased, higher loads were required to cause shear displacement. The confining load within the element at the rockmass and face plate interface had a much lower initial stiffness gradient within a higher pre-tensioned element.

• Use of strain-gauged rockbolts indicated a positive axial load within the reinforcing element, which was maintained consistently through the element until failure. Due to issues associated with installation and positioning of the strain-gauged rockbolts, many of the results were inconclusive, in regard to load distribution along the rockbolt.

• Failure of the reinforcing elements was in a typical ductile manner. Failure initiated in the centre of the necked region of the element with cracks initially outwards towards the surface of the element. The fracture was completed via a shear lip on the outer extremities of the element.
When the orientation of the strain gauges was aligned with the shear plane, failure of the rockbolt was initiated in one of the longitudinal slots at a location where there was maximum bending stress. This area was where the plastic hinges are created once a shear load was applied to the reinforced sample and leads to rockbolt failing offset the shear plane.

Future Actions

The outcomes from this stage of the project include:

- An understanding of the current knowledge related to the performance of reinforcing elements subject to shear loading
- The development of a controlled environment, laboratory test facility to analyse and isolate factors that influence the shear performance of reinforced rockmasses
- An initial series of controlled laboratory experiments using the test facility

Further testing and verification is required specifically with regard to:

- Borehole and element geometry
- Element orientation relative to discontinuity
- Element and encapsulation material geomechanical properties
- Block geometry
- Further element pre-tensioning
- Characteristic of discontinuity
- Discontinuity aperture

Further theoretical, mechanistic and computational studies are required to support the experimental program with the final objective to prepare a set of industry guidelines for the application of the reinforcement systems in discontinuous materials, with respect to their performance in shear resistance.

Recommendations have been made to further control the laboratory experiments and minimise any variations in the results, including manually
installing the reinforcing element to reduce the variability arising from drilling and installation.
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1. Introduction

Rock support has evolved within the mining, tunnelling and civil industries to the widespread use of rockbolts to reinforce and support the surrounding rockmass. New applications and innovations of rock reinforcement have continued to appear on the market and are regularly trialled and used throughout Australia and the rest of the world. Within Australian underground coal mines, rockbolts are the most utilised form of reinforcing elements for primary roof and rib support.

A typical Australian underground coal longwall mine producing around 3 million tonnes per annum, uses between 4,000 and 6,000 rockbolts per month, which equates to a total cost of approximately A$150,000 per month for rockbolts, plates, resin and accessories. The higher producing coal mines, found commonly in Queensland, Australia, use up to 7,000 rockbolts per month with increased reinforcing costs up to A$200,000 per month for all rockbolts and accessories.

The use of rock reinforcing elements is a very useful and flexible method to provide active support within the surrounding rockmass by imposing a reactive load through tensioning of these elements. These reinforcing elements are installed into drilled boreholes by anchoring and securing the element within the borehole and to the collar of the borehole by the use of a steel plate and nut, this is commonly known as rockbolting. Rockbolting is the most common and oldest form for rock reinforcing used throughout the world.

The term rockbolting was then developed ‘generally to indicate any form of mechanical support that is inserted into the rockmass with the primary objective of increasing its stiffness and/or strength with respect to tensile and/or shear loads’, (Gerard, 1983).
Rockbolts are commonly installed at a density of four rockbolts every 1.5 m in typical stable ground conditions and in relatively unstable roof the density is increased to six rockbolts per metre that can increase up to eight rockbolts per metre. In some instances cablebolts are installed in conjunction with the rockbolts in the unstable ground conditions.

In addition to pure tensile loading, rockbolts are also exposed to shear forces present in the underground mining environment. These shear forces can influence the overall performance of the reinforcing elements and have been examined, analysed and tested since the 1970s.

1.1. Research Objectives

The objectives of this research project are aligned with initial objectives of the ACARP project C12010 ‘Mechanical Behaviour of Reinforcement Elements Subjected to Shear’.

The objectives of this research project were:

1. To research the current understanding of the performance of reinforcement elements in shear
2. To design and develop a testing rig which meets the need of the required testing, and
3. To conduct a series of controlled laboratory experiments using the facility and to study the effect of the following variables on the performance of reinforcement elements in both direct shear resistance and indirect shear resistance through axial clamping:
   - Effective shear capacity of a support element
   - Influence of the loading rate on test outcomes
   - Test variations due to encasing material properties
   - Contribution of element pre-tensioning to shear capacity
   - Failure mode of the reinforcing element
1.2. Research Methodology

The methodology adopted for the project was as follows:

- Review research since the commencement of the project in June 2003, including the literature survey that finalises a parameter listing for analysis in this project and the published paper on the findings of the research of the last 30 years; Paper title "Understanding the Performance of Rock Reinforcement Elements under Shear Loading through Laboratory Testing: A 30-year History", by Hartman & Hebblewhite, 2003.

- Further extend all previous research and literature findings.

- Design, construct and commission a laboratory testing facility suitable for full-scale testing of shear performance by converting an old Avery-Denison compressive machine that is located in the School of Mining Engineering, University of New South Wales.

- After the facility was built and commissioned, reinforcement elements were installed into the specimen as the current industry practice and tested to determine the capabilities of the shear testing facility and further the influencing shear parameters within rock reinforcing.

- Use all tests in the testing program to dynamically modify and alter the shear testing facility to improve its capabilities and possibilities.

- Report and record all findings and conclusions with recommendations reported for continuation of the ACARP project.
1.3. *History of Rock Reinforcing in Australia*

The technique of roof bolting was first publicised in Australia (1908), reporting on rock bolting as a form of rockmass stabilisation in USA mines, but was not adapted in Australia as a method of roof reinforcement until the late 1940s. One of the first instances of the introduction of rockmass reinforcing using rockbolting techniques into Australian mines was in 1949 at Elrington Colliery. McKensey in 1952 studied and recorded the comparative success of roof bolting at Elrington Colliery.

Figure 1 shows an Elrington Colliery roadway prior to rockbolting, while Figure 2 displays similar roadways but where rockbolts were used as roof reinforcement (Martin et al, 1993).
The adoption and experimentation of rockbolts was necessary due to the introduction of mechanised mining equipment such as the first Joy 1CM continuous miner in 1950 at Newstan Colliery that increased production up to 300 tonnes per shift (Martin et al, 1993). The increase in production rates increased stability problems to the underground workings due to the limitations of the traditional timber chock and prop support systems. These traditional support regimes were time consuming to install and were closely spaced to achieve a high density supporting method. The advantages of rockbolts included lower cost, quicker installation and relatively concealed rockmass support which led to rockbolts being adopted in conjunction with the introduction of continuous miners into Australian coal operations (Gardner, 1971).
Without significant scientific and engineering understanding of the performance and mechanics of rockbolts, the New South Wales (NSW) Department of Mines in the early 1950’s expressed caution and distributed warnings that rockbolting only be undertaken in conjunction with normal levels of timbering. Full scale rockbolting was initiated at Elrington Colliery in April 1950 and consisted of 1” (25.4mm) diameter, 7ft 6” (2.3m) long, screwed 6” (152mm) at one end and split 9” (228mm) at the other end type of rockbolt. The initial support pattern was rows of three bolts, approximately 3 ft (0.9m) apart across the width of the roadway with 5 ft (1.5m) spacing between rows. This pattern was used in conjunction with the colliery’s standard timber support system. This pattern was widely implemented with a notable exception at Hebburn No.2 Colliery which adopted a rectangular pattern with rows of four bolts, 4 ft (1.2m) apart at a spacing of 4ft 6” (1.4m) as seen in Figure 3.

Figure 3 – Roofbolted transport roadway in Hebburn No.2 Colliery (after Martin et al, 1993)

In 1950 BHP began to introduce rockbolts into the northern and southern coalfields of NSW. The mobile miner was introduced from USA in 1954 to
Australia which resulted in the declining use of the slot and wedge type bolt and favour for the expansion shell anchor type elements. The mobile miner used rotary drilling methods which allowed for the tensioning of the bolts to be done by the bolter rather than by hand methods, as required with the slot and wedge type system. In 1955 the Titan Manufacturing Company Pty Ltd. commenced commercial manufacture of expansion shell bolts (Offner, 2000).

The development of the Snowy Mountains Scheme Project in 1949 was highly dependent on rock reinforcing elements in the form of rockbolts to provide adequate support. It was seen to be a safer and cheaper method compared to concrete lining throughout the project. This project led to increase scientific knowledge and understanding of the mechanism of rock reinforcing and paved the way for increased use of rockbolts within the mining, tunnelling and civil industries.

A polyester resin mixture that was developed in Germany was installed and tested at a number of NSW coal mines in the 1960’s. It was later concluded through tests by Barnes and Howe in 1964 that the resin encapsulated anchorage system displayed a superior performance over that of the mechanical anchors. The 1970’s brought the introduction of commercially economic chemical bolting systems into the Australian mining industry where the higher performance of resin anchorage in many bolting applications was recognised (Offner, 2000).

With a high expectation of safety and excavation stability in mined openings, continuous studies, experiments and innovative development of reinforcing elements has provided an increased understanding of the behaviour a rock-bolted rockmass and the load transfer between the element and rock. Rockbolting in Australia has dramatically improved since its first introduction in the late 1940’s and the current technical support, understanding and adoption to alternative conditions has continued to keep the Australian mining industry a leader in respect to innovation and comprehension of reinforcing elements.
1.4. **Terminology and Definitions**

The terminology that is used across the mining, tunnelling and civil industries is often interchanged and commonly has alternative meanings among the different industries and even within each specific industry. Especially the terms and definitions of ground control, support and reinforcement is often confused, if not mistaken.

1.4.1. **Rock Reinforcement**

Rock reinforcement is essentially seen to be an overall improvement in the rockmass properties from within the rockmass. This definition therefore includes all techniques and devices installed within the rockmass such as rockbolts, cablebolts and ground anchors. The installation of these reinforcing elements in a rockmass is intended to mobilise the rock’s natural competency to support itself (Brown, 1999).

1.4.2. **Rock Support**

Rock support is the application of a reactive force against the boundary surface of the excavation and includes techniques and devices such as backfill, timber props and stills, steel or concrete sets, shotcrete etc. The placement of rock support including wood sets, steel sets or reinforced concrete lining provides resistance to the movement of the surrounding rockmass towards the open excavation (Brown, 1999).
1.5. Types of reinforcing elements

There are numerous forms and designs of rock reinforcing elements in use to stabilise rock excavations, but the most common include rockbolts, cable bolts and ground anchors. The difference between rockbolting, cable bolting and ground anchoring is the increase in length and capacity of each of the reinforcing elements.

1.5.1. Rockbolts

Rockbolts are the most common use of rock reinforcement throughout the world and continued to help create stable excavations. Rockbolts come in numerous forms and designs that also boast numerous methods to fix them into the rockmass. Essentially, a rockbolt is an elongated, usually cylindrical, structural element usually of steel that is fixed into a borehole either using a mechanical point anchor or grouting agent to form a mechanical coupling or by an interference fit to form a frictional coupling. Although there are many variants to these basic principles, rockbolts are commonly described under the headings of mechanically coupled and friction coupled. (Windsor & Thompson, 1999).

1.5.1.1. Mechanically Coupled Rockbolts

Mechanically coupled rockbolts are anchored into the surrounding rockmass using a cementing agent or mechanical anchor. These rockbolts are either anchored into the borehole over their entire length in one procedure or installed with a mechanical anchor or expansion shell at one end, then subsequently encapsulated along the entire length of the rockbolt. Encapsulation of a rockbolt is usually undertaken with either a cementatious or resinous mixture that is inserted into the borehole. Resin capsules are inserted prior to the rockbolt. The latter is then rotated to rupture the capsule mixing the resin mastic and the catalyst producing a hard, solid encapsulation, bonding the rockbolt to the
surrounding rockmass. Cement based grout is one of the oldest forms of encapsulation and bonding. It is pumped into the borehole and set at relatively high strengths. Details on the load transfer, components and properties of mechanically coupled rockbolts will be discussed further in later sections.

### 1.5.1.2. Friction Coupled Rockbolts

One of the most popular and widely used forms of rock reinforcement in the hard rock mining sector is the friction stabiliser bolt. Friction stabiliser bolts are based on the theory of applying full contact with the walls of the hole and anchoring to the surrounding rock by forms of compressive force. This force is induced immediately and is one of the major advantages of the friction stabiliser bolt, especially when instant support is required after the excavation has taken place. Frictional stabiliser bolts are classified as a short-term support solution.

Dr. James J. Scott first invented friction stabiliser bolts in 1973, and in conjunction with Ingersoll-Rand Company, developed this rock reinforcement method for use in the mining industry in 1977. Ingersoll-Rand then marketed the rockbolt as the Split Set. All involved in the tunnelling, construction and especially mining industries soon grasped and welcomed the concept of the friction stabiliser bolt, due to its low weight, ease of installation and compatibility with current automatic drilling and bolting machines. A friction stabiliser bolt can be seen in Figure 4.

![Friction Stabiliser Bolt](SCS_2002)

Figure 4 – Friction Stabiliser Bolt (SCS, 2002)

Friction bolts exert prestressing forces perpendicular and parallel to the axis of the installed bolt. Passive rock bolts, such as non-tensioned grouted bolts do
not exert prestressing force on the surrounding rock and mechanical point anchor bolts exert a prestressing force solely about the axis of the bolt.

1.5.2. Cablebolts

A conventional cablebolt is a flexible tendon consisting of a number of steel wires, wound into a strand, and grouted into a borehole (Hutchinson and Diederichs, 1996). Cablebolts have been widely adopted in the mining industry after they were first introduced in the 1960s. Cablebolts are encapsulated mainly with grout but the use of resin is also becoming more widely accepted. Similar to the rockbolt, the loading capacity of the cablebolt is transferred from the steel tendon, through the encapsulation and into the surrounding rockmass. Lengths of cablebolts commonly range from 3 metres to 20 metres in the Australian mining industry. Cablebolting is effectively used in pre-reinforcement of areas that are to be excavated or currently open in order to increase the rockmass stability; effectively creating a safer work environment.

Cablebolts have developed considerably since their introduction from an original shaft winder rope and modifications to the design for increased load transfer has seen alternative designs being introduced into the market. One of the most significant changes to the cablebolt was the introduction of the ‘birdcage’ or ‘bulb’ within the length of the reinforcing element. This ‘birdcage’ is created by unwinding the strand and leaving an open-weave cross-section within the cablebolt as shown in Figure 5.
The increased use of the cablebolt is due in part to the improvement in the automatic installation, including drilling, handling, insertion and grouting. Cablebolts can be pre-stressed, or tensioned from a fixed anchor, to provide an active supportive force into the surrounding rockmass.

### 1.5.3. Ground anchors

Ground anchors are more commonly used in the field of civil engineering rather than mining. It is used to transmit an applied tensile load to a load bearing stratum. It commonly consists of an anchor head, free anchor length and fixed anchor as shown in Figure 6. Ground anchors are generally greater than 15 metres in length and tend to be arranged with large cross-sectional areas providing the large capacities needed to handle the large volumes of unstable material that require reinforcement. Commonly ground anchors are formed into two different categories based on their primary modes of action including high axial capacity elements and high shear capacity elements (Windsor & Thompson, 1999).
High axial capacity elements make up around 90% of all ground anchors and include an array of long individual elements that are orientated for a stable reinforcing element and discretely coupled over a fairly long anchorage length at the far end (bond length). At the collar of this reinforcing element, the ground anchor is secured to the rockmass face using an external mechanical fixture (free length). The free length is available for pre-stressing and enables considerable load to be dispersed around the surface of the rockmass through large, rigid stressing blocks at the free surface.

High shear capacity ground anchor elements are usually in the form of universal beam steel sections, large diameter steel tubes or railway line that is cast in concrete to continuously couple the element within the rockmass. These reinforcing elements are commonly used for pre-reinforcement in surface excavations. These type of ground anchors are installed sub-parallel to the
excavation boundary and provide a high shear resistance to any displacements that occur in the surrounding rockmass (Windsor & Thompson, 1999).

1.6. Research Limitations

During the past 30 years of testing and research on the shear performance of reinforcing elements, there is a general agreement in regards to the specific parameters that affect the shear performance of rock reinforcement elements. There seems to be a lack of understanding of the actual reinforcement installation method and the associated loading mechanisms that may cause failure. Especially of concern is the effectiveness of pre-tensioning, axial tensioning and/or normal loading (Hartman & Hebblewhite, 2003).

Shear testing of rockbolts was first reported at the Swedish Rock Mechanics Research Foundation in 1974 in hard rock. This has subsequently followed by a series of other research attempts around the world. The factors considered included the size (length and diameter) of the rockbolts, number of rockbolts, the inclination of the rockbolts, the relative displacements in joints, joint roughness, the effect of compression, relative strength of rock and grout and elastic modulus of rock and grout. Analytical and numerical solutions have been proposed based on these experiments.

Numerous testing facilities have been designed and developed over the years and tend to operate on a small-scale.

This thesis is part of a larger initiative in designing, developing and commissioning a laboratory testing facility suitable for full-scale testing of shear performance of reinforcing elements sponsored by the Australian Coal Association Research Program (ACARP). Initial design of this facility would have to best represent and create the issues that are common in the underground mining environment. The facility should be capable of isolating several variables in order to:
1. Research the current understanding of the performance of reinforcement elements in shear

2. Design and develop a testing rig which meets the need of the required testing, and

3. Conduct a series of controlled laboratory experiments using the facility, and to study the effect of the following variables on the performance of reinforcement elements in both direct shear resistance and indirect shear resistance through axial clamping:
   - Effective shear capacity of a support element
   - Influence of the loading rate on test outcomes
   - Test variations due to encasing material properties
   - Contribution of element pre-tensioning to shear capacity
   - Failure mode of the reinforcing element
2. Review of Previous Research

Shear movement in an underground excavation is a common occurrence especially in bedded strata with differential properties resulting from the insitu stresses. The roof of the excavation acts as a single beam of rockmass and deforms similarly to a steel construction beam. If the surrounding strata is made up of material with different mechanical properties then shear forces will result and can be present close to the surface of excavation. When the overlying strata deforms or sags into the excavation, the plies pass over one another and shearing will occur between the strata.

The level of shear strength between laminations can be determined by equation 1 (UNSW, 2004b):

\[ \tau = C_o + (\sigma_n \times \mu) \]

where:
- \( \tau \): shear strength
- \( C_o \): cohesion
- \( \sigma_n \): normal or clamping stress
- \( \mu \): coefficient of friction

Cohesion between rock layers is often negligible and the coefficient of friction is also commonly found to be close to zero.

Installation of reinforcing elements increases the resistance to shear by:

- Increasing the normal or clamping force with the tensioning of the elements, and/or
- Increasing cohesion between the rock layers.
The contact strength of the rock is also an important factor to consider once shear movement is initiated and the cohesion, normal stress and coefficient of friction are exceeded. Shear movement between layers of rock can occur by:

- The ply separating and riding over one another, and/or
- The contact strength of the material at the two interfaces being exceeded and failure surfaces developing within the rockmass

Maximum shear stress between rock layers usually occurs above the roadway and is very close to the abutments. Little shear is developed along the centreline of an excavation.

Reinforcing elements are consequently used to prevent or resist shear movement. The ways that these reinforcing elements achieve an increased overall resistance to shear movement is by (UNSW, 2004a):

- Resisting pure shear through the actual shear strength of the steel in the reinforcing element and the encapsulation medium
- Pre-tensioning the reinforcing element will increase cohesion and further mobilise friction and friction if contact can be re-established
- Preventing initial deformation by maintaining cohesion between adjoining layers of rockmass
- Increased tension through the reinforcing element due to shearing over a finite length
- Irregular surface can mobilise the rock shear strength and can further increase the tension in the reinforcing element through dilatancy

The earliest known shear testing was by Sten Bjurstöm in 1974 and consequently developed further with an increased analysis and testing by Charles J. Haas in the late 1970s and early 1980s. Simple direct shear experiments were arranged. It was soon understood that there was a rife of
parameters that have an influence on the overall shear performance of reinforcing elements.

While there are certain applicable standards for shear testing of reinforcing elements in other countries such as the United Kingdom and Germany, there are no such standards that exist in Australia.

2.1. **Rock reinforcement load transfer mechanisms**

The interaction between the reinforcing element and the surrounding rockmass varies among different element/rockmass materials and can be also very complex. The uncertain nature of the surrounding rockmass and the dynamic interactions that develop between the elements can alter the load transfer mechanisms throughout the life of the reinforcing element. In order to understand the interaction between the rockmass and the reinforcing element it is first necessary to understand two basic principles (Windsor & Thompson, 1999):

1. The load transfer concept for reinforcement systems.
2. The principal components of a reinforcing system.

2.1.1. **Load transfer concept**

Load transfer is induced when interaction occurs between the reinforcing element and the surrounding rockmass (via an encapsulated medium). Reinforcing elements may react with the surrounding rockmass either actively or passively depending on the installation procedure and capabilities of the actual reinforcing element. Basically there are three basic mechanisms in the load transfer concept and this is shown schematically in Figure 7 (Windsor & Thompson, 1999):

1. Rock movement which requires load transfer from the unstable rock to the reinforcing element.
2. Transfer of load via the reinforcing element from the unstable surface region to a stable interior region.
3. Transfer of the load from the reinforcing element to the stable rock mass.

![Diagram of load transfer concept](image)

**Figure 7 – Load transfer concept (Windsor & Thompson, 1999)**

### 2.1.2. Components of a rock reinforcing system

A reinforcing system has four basic components that interact through one another in order to produce a stable rockmass, these being:

1. The rockmass
2. The reinforcing element
3. The encapsulation (grout or resin) or mechanical anchor
4. External plate and nut

A schematic of the principal components can be seen in Figure 8.

The behaviour of the reinforcing system is a combination of each of the four components and their interaction with one another. Each component is involved with a two load transfer interaction but some systems may not include the encapsulation or mechanical anchor interaction between the reinforcing element and the rockmass, such as frictionally coupled reinforcing systems.
2.1.3. Load transfer interactions - Axial

This project investigated the performance of continuously mechanically coupled reinforcing elements subject to shear loads rather than other common types of reinforcing elements such as frictionally coupled systems and further examined the loading mechanisms of this reinforcing system.

Continuously mechanical coupled systems rely on encapsulation between the rockmass and the reinforcing element. The encapsulation material is either a cementitious grout or a resin product. This material is inserted as a fluid material and hardens or sets according to the material properties and requirements. The major function of the encapsulating material is to provide a mechanism for load transfer between the rockmass and reinforcing element.

Continuous, mechanically coupled systems can also be point anchored by means of an expansion shell or similar anchor. For an end-anchored system the rock between the anchor point and the rockmass collar is in compression. Any dilation of the rockmass between these two points will result in an increase in the compressive stress in the rockmass.

Fully encapsulated reinforcing elements are more complex in their element/resin/rockmass interaction and pullout tests have reported that the distribution of the bond stress along the element is an exponential function of the length of the element and that the maximum bond stress occurs at the free end before any slip takes place.

The transfer load between the rockmass, encapsulation material and reinforcing element must occur through an axially directed shear stress across any interfaces that exist between the element and the rock. The interfaces that are involved in a load transfer system can occur across a relatively smooth or rough interface, which can be seen in Figure 9 (Windsor & Thompson, 1999).
Figure 9 – Load transfer across relatively smooth and relatively rough mechanically coupled interfaces (Windsor & Thompson, 1999)
The strength of the reinforcing element is based on the interaction between the rockmass, encapsulation material and reinforcing element. This controls the load transferring capacity of the entire system. All interfaces are characterised by ‘interface zones’ that have the potential to reduce load transfer (Windsor & Thompson, 1999).

The significance of a pull test on the free end of the element is that a higher shear stress will develop at the free end of the bolt. The shear stresses some distance along the bolt will be lower. Since the shear stress at the bolt/grout interface is linked to the axial stress in the bolt it follows that the axial stress in the bolt will decrease with distance from the free end of the bolt. In areas where the shear stress reaches the shear strength of the grout, local debonding will take place and the maximum shear stress will migrate along the axis of the bolt (UNSW, 2004b).

A joint opening within the axis of the reinforced borehole imposes a similar load transfer response as to a pulling the element at the free end. For encapsulated elements, the resultant stress distribution along the length of the element would be quite dynamic and asymmetrical. Increased areas of stress will develop where joints open up and the stresses away from the joints will be low. At the direct location of the joint, the local stiffness of the reinforcing element will be very high since the length of the reinforcing element will be very short.

For a loading condition that is displacement controlled, the load that is generated by the reinforcing element depends on the stiffness of the element and the rockmass strata displacement. The stiffness of the reinforcing element, $k$, is given by equation 2 (UNSW, 2004b):
\[ k = \frac{A \cdot E}{l} \]  

(2)

where:

- \( A \) : Cross sectional area of the reinforcing element
- \( E \) : Modulus of elasticity of the element material
- \( L \) : Length of element over which the strata deformation is dissipated

For the case of end-anchored reinforcing elements, \( l \) is equal to the total length of the element (\( L \)). For encapsulated reinforcing elements \( l \) is a function of the strata displacement and the shear strength of the contacts between the bolt and the encapsulating material and also between the encapsulating material and the borehole wall.

### 2.1.4. Load transfer interactions - Shear

Reinforcing systems are primarily based on their axial strength and stiffness with little investigation or application in regards to the shear performance of the reinforcing elements. This is due to the lack of understanding in the mechanics involved with the reinforcing system exposed to shear forces and complexity of attaining shear performance data relative to axial strength of the reinforcing system.

The load transfer interactions of a reinforcing element under shear loading conditions includes all factors mentioned in the axial load transfer interactions but additionally the crushing of the rockmass/encapsulation material around the bore hole wall due to insitu stresses and shear/axial deformation of the reinforcing element as shown in Figure 10.
a) Schematic of shear movement along a reinforced surface

b) Shear movement along a reinforced angled joint surface (UNSW, 2004b)

Figure 10 – Schematic of a reinforcing element subject to a shear load

An end-anchored reinforcing element will provide minimal initial resistance to shear movement other than the resolved component of axial load and stiffness. It is not until the element is displaced to either extremities of the borehole wall that the majority of the shear resistance will take place (Figure 11). This
reinforcing system therefore provides little initial shear resistance until significant displacement has occurred and the entire system stiffens considerably.

![Figure 11 – Mechanism of resisting shear movement (UNSW, 2004b).](image)

A continuously coupled reinforcing element that intersects a discontinuity and is exposed to a shear force will provide immediate resistance to prevent the shear displacement. The shear stiffness and the peak shear capacity will depend on the cross-sectional shape of the element and the strength and diameter of the reinforcing element (Windsor & Thompson, 1999).

Very few reinforcing elements are subject to pure shear or axial loading individually. The loading conditions are a combination of both the axial and shear loading. The bending properties of the reinforcing element and the properties of the encapsulation/rockmass in response to crushing at the borehole walls are both very important parameters when the system is exposed to these shear forces. A physical representation of the reinforcing element undergoing deformation at a continuity subject to shear force can be seen in Figure 12.
2.1.5. Theories on Shear Behaviour

Many laboratory experiments have been undertaken to better understand the mechanics, load transfer and appreciation of reinforcing elements used within the mining, tunnelling and civil industries. The understanding of how the reinforcing element supports and reacts to the surrounding rockmass has led to innovative and functional elements to independently suit different mining conditions. The importance of these laboratory experiments is to practically analyse the performance of the reinforcing elements and to isolate and record the importance of the influencing parameters. An increased understanding of the forces that are subject to reinforcing elements has led to an increase in awareness of shear loading on these elements.
Hass (1976) was one of the first researches to develop a full-scale shear testing facility to simulate a shear loading condition of a reinforced rockmass. The shear testing facility was primarily two large rock blocks that were exposed to both a normal and shear force and reinforced with conventional and resin-grouted reinforcing bars. Even though this research and shear testing facility was a strong base for ongoing studies in this area, there were numerous failures associated with the tests as outlined below (Hass, 1976):

1. Either the bolt sheared or the left block split.
2. In several tests at the low normal pressure, particularly those with inclined bolts, the blocks separated to such an extent that the left side plate began to bear on the testing machine columns.
3. Vertical splitting occurred when the bolt acted as a wedge against the hole.
4. A larger bearing plate was placed on top of the left shear block to better distribute the shear load. Splitting of the left block still continued, but at a somewhat reduced frequency, particularly in tests involving high shear loads.
5. Bolts were either sheared off completely or greatly deformed. Shearing was more common at the high normal pressure.
6. At the low normal pressure the bolt deformed and forced the blocks to separate.
7. Crushing occurred in the surrounding rock to a depth of about 0.5 in. at both normal pressures since extremely high compressive loads were applied to the sides of the drill hole by the bolts and grouted bolts.

After numerous laboratory, empirical, analytical and numerical analysis on the shear performance of reinforcing elements, there is an agreed understanding that a reinforcing element subject to shear load deforms as the joint displacement increases as shown in Figure 13. At the intersection between the reinforcing element and the joint, a force $R_o$, is mobilised in the bolt. This force results from the axial force $N_o$, and the shear force $Q_o$, acting on the bolt. The reinforcing element then strengthens the joint surface by adding cohesion and
increasing confining stress by the normal component of the reinforcing element (Pellet & Egger, 1996).

Figure 13 – A bolted rock joint subjected to shearing (Pellet & Egger, 1996)

Many equations have been developed and derived from analytical studies to predict the behaviour of reinforced rock joints subject to shear loads especially considering the deformation of the element subject to this load. Elastic beam theories were introduced and adapted into predicting this particular behaviour but are very limiting only to the initial stages of the loading process. Failure of the reinforcing element is determined by the combination of the axial and shear forces, whereas displacement is calculated by taking into account the yields of the encapsulating material (Pellet & Egger, 1996).

Pellet and Egger in 1996 further discuss the importance of properly modelling the behaviour of the reinforcing element. Important factors to consider in understanding and predicting the effectiveness of reinforcing elements subject to a shear load include (Figure 14):

- The yielding and plastic strain of the reinforcing element
- The shear force mobilised in the element
- The stiffness of the element
Grasselli (2004) developed a large-scale shear testing facility in order to better understand the mechanical responses of fully grouted steel bars subject to a shear load. Two reinforcing bars were installed into concrete blocks and analysed through the double-shear testing method. Grasselli analysed the results through the reinforcing bar contribution that he defined as $T^*$ and calculated $T^*$ as the difference between the applied shear force, $T_V$, and the frictional strength provided by the unbolted smooth joints, $N \tan\phi$, normalised to the ultimate tensile load of the bolt $F_{\text{max}}$ and shown in equation 3:

$$
T^* = \frac{\text{bolts contribution}}{2nF_{\text{max}}} = \frac{T_V - 2N \tan\phi}{2nF_{\text{max}}} \quad (3)
$$

The observation and experimental curve of the fully grouted steel bar is shown in Figure 15.
The first section of the graph corresponds to linear behaviour, small displacements and a high increase in load. The full steel bolt is shown to mobilise 75% of its resistant contribution with only 1.8mm of shear displacement. The second part of the curve displays the yielding of the material with formation of plastic hinges with the progressive failure of the grout around the rock over 8mm of displacement. Finally, the third section of the curve corresponds to a nearly unconstrained plastic deformation of the bolt, until failure. Failure of the bolt is primarily caused by the traction load concentrated between the two plastic hinges (Grasselli, 2004). Figure 16 shows the shape of the steel bolt and the plastic hinges that are formed prior to failure.
Failure of the fully grouted steel bar is caused by growth of two plastic hinges symmetrically to the rock joint and increase in stress in these areas as displacement continues. After the formation of these plastic hinges, creating a zone of plastic deformation, remains until complete failure of the steel bar (Grasselli, 2004).

2.1.6. Basic mechanics of roof support

Reinforcing a solid rockmass using reinforcing elements is intended to keep the excavation open and create a safe productive underground environment. Reinforcing elements basically prevent strata separation and uncontrolled roof falls by either maintaining or increasing the strength properties of the jointed rockmass through mobilisation of frictional forces.

The main forms of basic roof support mechanisms can be categorised and described as:

- Suspension – of a thin layer of strata
- Beam building – clamping of strata layers
- Rock arch – formation of a load carrying arch
- Keying of blocks – supporting potential wedge failures

2.1.6.1. Suspension Mechanism

The suspension mechanism is one of the most widely known mechanisms and basically consists of supporting a thin section of overlying strata by installing reinforcing tendons through this layer and into a stronger layer of rockmass. The stronger layer of rockmass is seen to be relatively stiffer and the relatively weaker rockmass is pinned to this stiffer layer as seen in Figure 17.
Panek in 1962 explained how the increase in friction supplied by tensioned reinforcing elements reduces bending in a laminated roof environment. If the beds tend to have unequal deflection, the elements oppose this tendency and create the suspension effect. Friction effectively reduced the bending of the strata in the reinforced section, where suspension, which depends on flexural rigidity, increased or decreased the maximum outer fibre bending stress in a single layer.

If the suspension effect is substantial, because one bed is much thicker than the others and if the rockbolts are pretensioned, then the greatest bending stress is likely to occur in the thick bed. Should failure occur under these conditions, it will be initiated in the thick bed; consequently the bolted unit will fail en masse, rather than bed by bed (Panek, 1962). Figure 18 shows the suspension of loose roof slabs supported by reinforcing elements.
It is understood that the suspension effect involves supporting the weaker or detached layer as a dead weight. Snyder (1983) indicated that the reinforcing element transfers the dead weight load and the load induced by the displacement of the weaker strata into the relative stiffer surrounding rockmass. Fully encapsulated elements were seen to be the most effective in transferring the load of the detached layer of rockmass into the stronger layers. This was due to the fact that encapsulated elements provide a larger surface area for the transmission of the load to occur.

2.1.6.2. Beam Building Mechanism

The overlying strata, especially in the roadways of coal mines are commonly thinly laminated. These thin layers can deflect or sag into the roadway with the main purpose of reinforcing elements to increase the effective thickness of the immediate roof beam and make it self supporting. When the roof deflects, the upper beam is shortened and differential movement occurs between the layers.
leading to shear displacement taking place relatively between the top layer of the lower roof slab and the bottom layer of the upper roof slab.

Reinforcing elements are installed to resist this movement in the layers by increasing the frictional strength between the different layers. Fully encapsulated reinforcing elements are the preferred method to increase the frictional strength and reduce the movement between the layers by three main factors (UNSW, 2004b):

- cohesion that may exist between the two layers,
- friction between the two layer,
- normal (clamping force) that acts on the layers.

For an effective normal force to be applied to the layers the reinforcing element must be pre-tensioned in order to apply an active, clamping force and increase the frictional strength between the layers.

The beams that are developed in excavated roadways are described to have the same properties as gravity loaded beams that have both ends clamped and governed by equations 4 to 6 (UNSW, 2004b).

**Maximum Deflection (Displacement):**

\[
d_{\text{max}} = \frac{k_1 s^4}{E t^2} \tag{4}
\]

**Maximum Bending Stress:**

\[
\sigma_{\text{max}} = K_2 \frac{s^2}{t} \tag{5}
\]
Maximum Shear Stress:

\[
\tau_{\text{max}} = k_3 \ s
\]  

(6)

where:

- \( s \): Roof span (width of roadway)
- \( t \): Thickness of roof layer
- \( k_1, k_2 \text{ and } k_3 \): Proportionality factors for different beam and plate configurations

The shear stress in the beam is zero in the middle of the roadway and therefore the density of the reinforcing elements should be higher close to the abutments instead of the centre of the roadway.

Panek in 1956 compared the bending, displacements and mechanisms of a single and multiple layered beams graphically as shown in Figure 19.

![Figure 19 – Single and four member beams (Panek, 1956)](image)

Panek’s findings indicate that reinforcing elements must be pre-tensioned in order to clamp the beds together and therefore creating frictional resistance to
bedding plane slip and decreasing the flexure of the roof beds. Reinforcing elements that are not tensioned accordingly only support the surrounding rockmass through suspension (Panek, 1956).

2.1.6.3. Rock Arch

Reinforcing elements are widely used to support large excavations especially in jointed rockmasses. The support design is based on two main criteria. The first is to identify any major or potential unstable rock blocks and to reinforce these blocks accordingly. The second criteria is to establish a compressive arch in the crown of the excavation through installation of reinforcing elements as seen in Figure 20 (UNSW, 2004b).

Cablebolts are usually suited as a means of rock reinforcing due to the fact that a rock arch support is required to stabilise the large size of the excavations. As in the beam building effect, it is essential to tension the reinforcing elements in order to form the rock arch. Supporting the large excavation effectively takes the installation of large and short reinforcing elements to achieve the overall design objective and stability. Blocky ground also requires introduction of wire mesh and shotcrete to further stabilise the surrounding rockmass (UNSW, 2004b).
Rock arching is not restricted in only defining the formation of an arch or domed shape opening. The term is also used to describe the process by which fractured rocks and soils may become partially self supporting by the formation of a compressive zone above an opening. This zone transfers vertical load to the abutments on each side of the opening. An underground roof beam that is cut by transverse cracks will form an arch if the insitu forces acting on the beam are not sufficient to overcome the tensile stresses in the roof (Wright, 1973)

Wright (1973) also identifies how a cracked beam can generally fail:
1. If the horizontal thrust is not great enough, the blocks will simply slide down and the roof collapses.
2. The rock crushes at points of high compressive stress, permitting rotation of blocks and subsequent collapse.
3. Elastic or inelastic buckling where the rock at the abutments and centre deforms to such a degree that rotation of blocks can occur without exceeding the crushing strength of the rock at points of rotation.

Essentially the rock arch is created and formed to increase the stability of an excavated opening. Through the use of reinforcing elements a compressive zone is created and acts as a structural membrane providing local support under its own weight and also supporting the surrounding rockmass.

2.1.6.4. Keying of Blocks

Reinforcing elements are recommended to be used to support potentially unstable wedges or blocks which are free to fall or slide under their own weight. This is because these blocks can move independently with respect to the remainder of the rockmass and hence apply concentrated or eccentric loading to the support system as shown in Figure 21. To prevent smaller wedges or blocks also falling out from a closely jointed rockmass reinforcing elements are used in conjunction with wire mesh and straps. The frictional resistance of the
sliding surfaces of wedges and blocks that are free to slide should be taken into consideration when designing an appropriate supporting system (Offner, 2000).

Figure 21 – Keying action of rockbolts (Windsor & Thompson, 1993)

2.2. Failure mechanisms of reinforcing elements

Reinforcing elements that are manufactured for the mining industry are dominantly rolled from high strength steel. The steels that used to manufacture the reinforcing elements may fail in several ways and are influenced by a range of factors, but normally the steel fails in either a ductile or brittle manner. Ductile failure occurs after considerable plastic deformation is exhibited prior to ultimate failure, where brittle failure exhibits little or no plastic deformation prior to failure.

2.2.1. Steel Properties

All reinforcing elements whether rockbolts, cablebolts or ground anchors are manufactured from high strength grade steel designed for mining applications. The steel properties are important in understanding the mechanical and physical properties of the reinforcing element and determining the suitable application in a mining environment. The steel grade is also important in
determining the ultimate tensile strength of the steel. Other important physical properties that should also be considered include (Crosky et al, 2004):

- **Yield strength**
  - This is the stress beyond which permanent deformation of the bolt occurs

- **Elongation of the bolt**
  - This can be measured as *Uniform Elongation* which occurs over the whole length of the bolt under load, or can be quoted as *5D Elongation* which occurs over a length of 5 bar diameters about the final failure surface

- **Fracture toughness**
  - Fracture toughness is the capacity of the steel to resist brittle fracture when subject to an impact load.

### 2.2.2. Ductile Failure

Ductile failure of reinforcing elements is common with intact, negligible-defect rockbolts. For ductile failure of a steel rockbolt cracks must develop before the fracture of the reinforcing element can occur and the way that these cracks develop and propagate through the steel that controls the mode of failure. The failure of the reinforcing elements that are subject to an axial load fail in a ductile manner, which is caused by cracks developing slowly with a considerable amount of plastic deformation resulting in necking when the load is continued to be applied.

Typical ductile failure of a steel rockbolt occurs with the initial steep elastic response. With further loading the steel reaches a yield point, beyond which plastic deformation of steel occurs.
Fracture begins at the centre of the necked region on a plane normal to the applied tensile-stress axis. As deformation progresses, the crack spreads laterally towards the edges of the rockbolt with completion of the fracture occurring very rapidly along a surface that makes an angle of approximately 45 degrees in the tensile-stress axis, this final separation of the cross section is known as shear rupture (Reed-Hill, 1992) (Figure 22).

![Figure 22 – Cup and cone fracture. Crack starts at the centre of the specimen and spreads radially (Reed-Hill, 1992)](image)

In a perfect example, the final stage of a ductile failure to a rockbolt would leave a circular lip on one-half of the specimen and a bevel on the surface of the other half; this is commonly known as the cup and cone surface (Reed-Hill, 1992).

Most tensile fracture surfaces produce two or three visible zones classified by surface marks and characteristics during typical tensile fracture of steel. These three zones include the fibrous zone, radial zone and shear-lip zone (ASM Handbook Committee, 1961). The three zones are illustrated schematically in Figure 23.
The fibrous zone is a region of slow crack growth and is located in the centre of the fracture. This zone surrounds the fracture origin, which is usually situated at or near the tensile axis. The origin of fracture is centrally located in the fibrous zone and can usually be traced to a discontinuity.

The radial zone occurs with a change from slow crack growth to rapid or unstable crack propagation and radial marks then further develop in the general direction of crack extension. These diverging marks emanate either from the periphery of the fibrous zone or, in the absence of a fibrous zone, from the fracture origin itself. The radial marks that are developed converge to the point of origin.

A shear-lip zone is the final zone produced by fracture that consists of a smooth, annular area adjacent to the free surface of the rockbolt. In general, the size of the shear-lip is a function of the stress state and the properties of the metal (ASM Handbook Committee, 1961).
2.2.3. Brittle Failure

Brittle failure occurs commonly when defects or cracks propagate through the reinforcing element material caused by a variety of factors including corrosion, vibration, impact loading and bending. Brittle failure is characterised by fracture proceeded by a relatively small or negligible amount of plastic deformation or strain.

The steel’s ability to resist crack growth is not dependent so much on the yield strength, UTS or the elongation, but on the fracture toughness of the steel. Brittle failure can either occur because (Crosky et al, 2004):

- The steel grade itself is brittle (typical with high tensile strength, high carbon steels), or
- Ductile steels contain cracks or defects

Brittle failures have certain characteristics that permit them to be identified as follows (Crosky et al, 2004):

- There is no gross permanent or plastic deformation of the metal in the region of brittle fracture;
- The surface of a brittle fracture is perpendicular to the principle tensile stress;
- Characteristic markings on the fracture surface frequently point back to the location from which the fracture originated. These markings are sometimes referred to as “chevron” or “herringbone” marks, and may originate from a small crack or defect in the bar or section.

The following main factors determine whether brittle failure will occur in metals or in particular steel reinforcing elements (Crosky et al, 2004):

- A flaw or defect in the metal such as a crack
- The grain size within the metal itself, generally a smaller grain size is less susceptible to brittle failure;
- The temperature as metals become more brittle at low temperatures and more ductile at high temperatures;
• A stress level sufficient to cause crack propagation;
• Metal composition;
• Metal fracture toughness

2.3. Factors influencing shear performance

The parameters that influence the shear performance of reinforcing elements and that have been studied through laboratory experiments can be grouped in the following sub headings (Hartman & Hebblewhite, 2003):

1. Rock Mass
   - joint opening/aperture
   - joint surface roughness
   - joint strength
   - dilatancy during shearing
   - properties of host rock
   - rock deformability vs. bolt deformability

2. The reinforcement element system
   - hole size
   - hole roughness
   - bolt diameter
   - type of grout
   - thickness of grout collar
   - grouted vs. un-grouted bolt
   - fully bonded vs. point-anchored bolt
   - bolt material, strength and its deformability
   - inclination of the bolt
   - tension force in the bolt
   - deformed length of the bolt
3. Loading conditions

- pre-loading of bolt (tensioning and torque)
- normal stress on the shear surface
- shear displacement-induced tension
- dilatancy-induced tension during shearing
- magnitude of shear displacement
- deformed length of the bolt

There have been numerous laboratory experiments undertaken to identify the influence of the above parameters. The following discussion will summarise the findings of the previous studies that have examined each of these parameters.

2.3.1. Rock Mass

Installation of reinforcing elements is intended to primarily increase the stability of the surrounding rockmass for safety and production issues. Rockmass can have a major impact on the reinforcing element in terms the effectiveness, mechanisms and therefore reinforcing capabilities of the actual element. Identical reinforcing elements are installed into rockmasses of different properties can produce different results. The influence of the rockmass has been examined since the introduction of reinforcing elements.

Rockmass is a major variable when determining the performance of the reinforcing element to stabilise and support an excavation. Prior to the implementation of a reinforcing element into a particular operation or environment, significant tests are undertaken to predict the performance and capabilities of the system in the unique geological environment. This itself reiterates the important influence that the rockmass has on the performance of the reinforcing system and the required understanding of the geological properties and mechanics of this rockmass.
2.3.1.1. Joint opening/aperture

Joint opening/aperture has considerable influence on the shear resistance of reinforcing elements. Appreciable opening of fractures prior to shear movements results in double bends and failure of rockbolts by tension (Spang & Egger, 1990). Bolts across unopened fractures had been actually cut (guillotined) during rock bursts (Hartman & Hebblewhite, 2003).

More recently, Strata Control Systems undertook controlled mechanical tests to determine the shear load of BBS 22-XHT steel bars using a jig manufactured by Strata Control Systems. The steel bars required a higher shear load of 260kN to shear the bar at 0mm clearance compared to a shear load of 230kN to fail at a clearance of 10mm. Tests were undertaken using a universal testing machine of capacity 1800kN (AMEC, 2003). A summary of the report can be seen in Table 1.

### Table 1 – Maximum shear loads to cause failure of BBS 22-XHT bars (AMEC, 2003)

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Clearance (mm)</th>
<th>Shear Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1190/1</td>
<td>0</td>
<td>255</td>
</tr>
<tr>
<td>1190/2</td>
<td>0</td>
<td>255</td>
</tr>
<tr>
<td>1190/3</td>
<td>0</td>
<td>260</td>
</tr>
<tr>
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<td>260</td>
</tr>
<tr>
<td>1190/5</td>
<td>0</td>
<td>260</td>
</tr>
<tr>
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<td>5</td>
<td>240</td>
</tr>
<tr>
<td>1190/7</td>
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</tr>
<tr>
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<td>10</td>
<td>230</td>
</tr>
</tbody>
</table>
2.3.1.2. Joint surface roughness

The joint surface roughness can change dramatically within the surrounding rockmass depending on the type of rock and its individual properties. The common classification of joint surface roughness is simply classified from smooth to rough. Barton et al. (1974) developed a rockmass classification system to provide an empirical means of characterising the rockmass and identify the influencing factors that may affect the stability of underground openings. Barton’s rockmass classification system was called the Q-System and formed a central part of the new Austrian Tunnelling Method (UNSW, 2004a). A major parameter for the Q-System classification is the joint roughness number $J_r$. $J_r$ describes the large and small scale surface texture of the critical joint set and ranges from discontinuous joints to rough-undulating up to slickensided-planar joint surface roughness.

Smooth shear joint surfaces are recommended to evaluate alternative variables in determining the influence of shear resistance on reinforcing elements. This is somewhat true in the underground mining environment when the shear plane is located between bedding planes or at a clay/mineral infill. At an induced natural fracture in the rockmass that creates a joint surface, the joint shear surface is commonly rougher rather than smooth. Behaviour of the natural fracture shear blocks can be seen in Figure 24.
Hass (1981) considered this occurrence in the research in the performance of reinforcing elements to resist shear movement and concluded that while the joint surface roughness has an initial influence in the resistance to shear, the experimental studies showed that the roughness and planarity of the shear surface had little effect on shear displacements over 63mm, when the normal stress is held constant. Interlocking of the asperities of the rough shear surface caused a high level of shear resistance at small shear displacements (less than 2.5mm) but then the coefficient of friction rapidly drops off and approaches the same influence as a smooth surface (63mm).

2.3.1.3. Dilatancy during shearing

Irregularities on the rough joint surface cause the shear joint to dilate as shear movement propagates along the plane and consequently dilatancy induces an
axial force into the reinforcing element. This dilation has a similar effect on the bolt as bed separation across a joint plane and leads to localised high reaction forces and can be seen below in Figure 25 (UNSW, 2004b). Hass in 1981 examined this influence and concluded that there was an increase in tensile loading very close to the shear surface.

![Image](image.png)

Figure 25 – Dilation due to shear movement (UNSW, 2004b).

2.3.1.4. Properties of host rock

The properties of the host rock have seen to heavily influence the resistance to shear of a reinforcing element. Schubert (1984) concluded that bolts embedded in harder rock require smaller displacements for attaining a given resistance than those in softer rock. Spang and Egger (1990) investigated shear resistance of blocks reinforced with 8mm bolts. The deformability of rock was shown to be an important parameter for shear resistance of bolted rock joints. The response was softer in weaker blocks since the rock was more easily crushed near the exit point. Higher reinforcement resistance was obtained in weaker blocks of
rock because the cable aligned itself with the load around the crushed borehole (Hartman & Hebblewhite, 2003).

Pellet and Egger (1996) explained that when the strength of the host rock is high, the joint displacement at failure becomes smaller and the maximum rockbolt contribution decreases slightly. The same influence of the host rock properties was examined by Ferrero (1995) in which he concluded that maximum shear resistance is obtained from a ductile reinforcing element material, installed in weak rock specimens. Stronger and stiffer host rock materials lead to a higher shear stress in the reinforcing elements and consequently a lower global resistance of the reinforced joint.

2.3.1.5. Rock deformability vs. bolt deformability

Schubert (1984) found that the deformability of the surrounding rock is important for the bolt reaction. Soft steel improved the deformability of the bolted system in soft rock. When the reinforcing element deforms and exerts pressure on the borehole wall, a localised zone of rock deformation develops (Figure 26, UNSW, 2004b).
Chappell (1989) concluded from an analytical approach that by considering rock bolt stiffness and its interaction with the rockmass, the rockmass deformational response is predicted and its anisotropy examined. It was observed that by increasing the stiffness of the joint controlled shear moduli, the rock mass’s deformational response and anisotropy are significantly reduced. Passive rockbolts and their distribution are shown to reduce anisotropy but to varying degrees, dependent on the relative joint and rock material stiffness’s plus their relative volumes. The rockbolt stiffness effect on a softer rockmass is not as great as that for a firm rockmass and therefore passive rockbolts need to be relatively closely spaced to be effective in a softer rockmass.

2.3.2. The reinforcement element system

Reinforcing the shear plane with a reinforcing element improves the overall resistance to shear of the rockmass. The reinforcing element works as additional resistance against shear failure along joints; hence the entire
rockmass becomes stronger and deforms less (Egger and Zabuski, 1991). The shear resistance of the bolted joint consists of its proper strength contribution (equation 7) and of the contribution of the reinforcing element. The contribution of the reinforcing element is a result of the rock, and depends, therefore, on the Young’s moduli of these materials as well as the mortar cylinder. According to Spang and Egger (1990) the maximum contribution of the bolt, $T_o$, to the total shear strength of the joint is a function of the parameters, as shown in equation 7.

$$T_o = T_u \left(1.55 + 0.011\sigma_c^{1.07} \sin^2 (\beta + i) \cdot \sigma_c^{0.14} \left(0.85 + 0.45 \tan \phi\right)\right)$$  \hspace{1cm} (7)

Where:

- $T_u$: ultimate axial bolt load (tensile strength)
- $\beta$: angle between the bolt and joint surface
- $i$: angle of dilatancy
- $\phi$: angle of friction along the shear plane
- $\sigma_c$: compressive strength of the host rock

Ferrero (1995) outlined how the reinforcing elements increased the rock mass resistance. The global (rock and steel) resistance is due to the steel deformation induced by the rockmass strain and the combination of two effects:

1. The increment of the axial force due to the bar deformation ($T_r$)
2. The dowel effect ($Q$)
The increase of the axial force acting in the bar, due to the relative displacement of the two sides of the joint determines two effects (Figure 27) (Ferrero, 1995):

1. the increase of the component of the axial force acting perpendicularly to the joint leads to an increase in the joint strength due to friction forces,
2. the increment of the component of the axial force acting parallel to the joint directly increases the joint resistance

The dowel effect represents the resistance due to the shear forces acting in the bar. The interaction between the rock and bolt depends on their deformability characteristics (Ferrero, 1995).

2.3.2.1. Hole size

In conventional pull-out tests, the strength of resin encapsulation is influenced by the annulus thickness of encapsulation. The influence of hole size has a major influence on an installed reinforcing element into a surrounding rockmass. Larger diameter holes can sustain larger shear stresses at the resin-rock interface, but peak system performance is achieved when the resin annulus is optimised. The annual size is usually defined as:
Annulus size = \frac{(\text{diameter}_{\text{hole}} - \text{diameter}_{\text{bolt}})}{2}

Increasing the hole diameter size without a corresponding increase in the reinforcing element diameter leads to a larger resin annulus size. In larger resin annulus sizes, the primary advantage is that the resin is more likely to be properly mixed during the installation process. Snyder et al (1979) stated that annulus thickness is a limiting factor, independent of the reinforcing element diameter. If a larger hole size is required to reduce the shear stress at the resin/rock interface than a larger bolt should be used to keep the annulus thickness at an optimum. It is found that a significant reduction in shear stress capacity with an increasing hole size and higher bond strengths and anchorage capacities can be realised with reduced annulus systems (Offner, 2000).

More recently Hagan and Weckert (2004) examined the effects of a change in resin annulus on the behaviour of the rockbolt. They found that the strength or load bearing capacity of fully encapsulated rockbolts was independent of changes in resin annulus where resin annulus remained less than 4 mm (Figure 28). With greater resin thickness there was a reduction in the magnitude of anchorage strength and an increase in the variability.
2.3.2.2. Hole roughness

The load carrying capability of reinforcing elements depends to a certain degree on the smoothness or roughness of the borehole. Results from Gerdeen et al. (1977) concluded that the smoother the hole, the less load carrying capability of the system. Randomly grooved holes achieved the highest load capacity, followed by drilled holes, then holes that were worked smooth.

Snyder et al (1979) also found that as the degree of surface roughness increased, the load transfer at the rock/resin interface increased. The smoother the borehole, the less the load carrying capability. Grooving in the hole increased the pull out force by a factor of two or three over the normal hole. It was also found that appreciable variation in the pull out force can be expected for holes of supposedly the same conditions. The amount of variation in pull out load decreased as the degree of hole roughness increased.
2.3.2.3. **Bolt diameter**

Shear resistance increases with an increase in the bolt diameter and consequently increases the shear deformation experienced. This is due to the increase in bolt tension and with it the increase in frictional resistance in the joint plane (Figure 29) (UNSW, 2004b).

![Figure 29 - Effect of bolt diameter on shear resistance and shear deformation of roof bolts of different bolt diameter (UNSW, 2004b)]
A comparison by Grasselli (2004) of 16 and 20mm diameter rockbolts show that when increasing the bolt diameter, there was an increase in rigidity of the reinforced system. This is most likely due to the increase in surface area of the single reinforcement (Figure 30).

Figure 30 – Influence of the diameter on the mechanical behaviour of reinforced joints (Grasselli, 2004)

2.3.2.4. Inclination of the bolt

Each reinforcing element is subjected to a combination of pure shear forces and axial loading. The critical issue and key component in determining the contribution to each of these forces is the orientation of the reinforcing element in relation to the joint plane. The orientation of rockbolts relative to the shear plane has a pronounced effect on the shear resistance offered by the rockbolts. The effect of the orientation/inclination of the reinforcing element in regards to shear resistance is an issue that has been well researched and analysed over the past 30 years.

Sten Bjurström (1974) was one of the first researchers to address the issue of the inclination of the bolt with respect to the shear surface. The shear resistance was increased when the bolts were inclined increasing the shear strength with
small displacements. Hass (1976) concluded that “conventional bolts orientated such that they lose tension as shear displacement occurs added no shear resistance since the tension drops to zero at very small shear displacements”. Looking further into this statement, Hass identified that bolts inclined from $0^\circ$ to $+45^\circ$ (measured from normal to the shear surface and in the direction of the applied force, see Figure 31) increased the overall shear resistance and that bolts inclined at $-45^\circ$ provided no additional shear resistance as the bolt lost tension at very small displacements. Rockbolts are most effective when they are inclined to the shear surface so that they tend to elongate and experience a higher axial force as shear progresses. The rockbolts are quite ineffective when they are inclined the other way so as to experience axial compressive loading (Haas, 1981).

Figure 31 – Schematic sketch of loading geometry showing section of rock, fracture surface and grouted bolt (Haas, 1976)

Egger & Pellet (1990) reported that bolted samples of concrete blocks were tested in a high capacity press and found that:

- The optimum angle of bolt inclination with respect to the joint was $30^\circ$ to $60^\circ$. 

58
Bolts perpendicular to the shear plane furnished the lowest shear resistance.
Shear displacements at failure were minimal for bolt inclinations between $40^\circ$ and $50^\circ$.

Grasselli (2004) has recently undertaken experimental tests in regards to reinforcing element orientation. Shear tests were undertaken and adopted using double bolted joints installed into three large concrete blocks, with the shear load applied to the system pushing the central block progressively downward (Figure 32).

![Experimental set-up adopted for the shear tests on double bolted joints (Grasselli, 2004)](image)

Figure 32 – Experimental set-up adopted for the shear tests on double bolted joints (Grasselli, 2004)

Results from the experimental test concluded that the variation in the bolt inclination ($\beta$) affects both the maximum load mobilised by the reinforcements and the rigidity of the jointed system (Figure 33).
2.3.3. Loading conditions

Past research has focused primarily on the surrounding rockmass, reinforcing type and method of installation. There seems to be a lack of understanding of the actual reinforcement installation method and the associated loading mechanisms that may cause failure. Especially of concern is the effectiveness of pre-tensioning, axial tensioning and/or normal loading.

2.3.3.1. Pre-loading of bolt (tensioning and torque)

Hass (1976) was one of the first researchers to address this issue but to a very limiting degree. In his experiments a torque was applied to the bolts after a normal load was applied to the block, though the reason not made fully clear. Albeit the low loads applied to the bolted rockmass resulted a significant increase into overall resistance and in applying this force the shear resistance increased significantly by 10 fold. However this increase is quite small when compared to the corresponding shear strength without a bolt (Hartman & Hebblewhite, 2003).
Case studies from the United States reported by McHugh and Signer (1999) indicated shear loading contributed significantly to the failure of bolts used for rock reinforcement in coalmine roofs. One of their main findings was that the axial loading had little effect on a joint’s resistance to shear loading. Recent analytical research in the performance of pre-tensioned bolts suggests that only about 10% of the torque applied to a bolt is converted to axial tension in the bolt, the remaining 90% is consumed in overcoming thread and bearing friction (Fernando, 2001).

Pretension is said not to influence the maximum resistance of the reinforced shear joint, but as it effectively determines a stiffer system (Figure 34) it heavily influences the magnitude of the shear displacements (Ferrero, 1995).

![Figure 34 – Laboratory shear test result diagrams for prestressed bars and dowels. On the horizontal axis shear displacements, on the vertical axis the ratio between the shear applied force and the bar tensile strength (M = F/T₀) (Ferrero, 1995)](image)

Current research and laboratory tests by Jalalifar et al. (2004) demonstrated that increased tensional bolt load, and hence increasing joint surface confining pressures, contributed to joint structural stiffness. This result was supported by their modelling results. A higher value of shear stresses was concentrated near the shear plane in the bolt and increasing the pretension load caused a reduction in shear load, reduced contact pressure and increased the bolt
resistance to shear. Beyond the yield point shear stress for all situations remained constant (Jalalifar et al., 2004).

### 2.4. Summary

The findings from the literature study indicated a general agreement among researchers and practitioners regarding the influence of some of the parameters involved but also indicated a minimal understanding of the effect of actual reinforcement installation method and associated loading mechanisms may have on failure. Especially of interest are the effectiveness of pre-tension, torque, axial tension and/or normal loading.

All previous experiments, analysis and reports still reach inconclusive results in understanding the performance of reinforcing elements subjected to a shear load across a jointed surface. In particular the main variables that require investigation include (Hartman & Hebblewhite, 2003):

- Is pre-tensioning of the reinforcing element across a jointed surface maintained until loading of the rockmass occurs?
- Is actual torque reducing the yielding strength of the reinforcing element, and subsequently the shear strength?
- What role does the size of the bearing plate play in the performance of the reinforcing element in shear loading?
- Will it be beneficial in upgrading the elastic moduli of grout (including resin)?
- Is the stiffness of the reinforcing element assisting or destroying the resistance to shear loading within the reinforcing system?
- What effect has shear loading on the surrounding rockmass?
- Will an Australian standard or guideline assist in proper design for expected shear loading conditions?
3. Design of testing facility

3.1. Purpose and Specifications

The principal objectives of this research project were to design and construct a testing facility to further understand the performance of rock reinforcement subject to shear loading. While there are several shear testing facilities in existence, they are based on guillotine testing of the reinforcing element, focusing on the shear strength of the actual material that the reinforcing element is constructed from.

Previous work has indicated that the mechanical properties of the rockmass has a dramatic influence of the shear strength of the reinforcing element and that a stiffer and harder rockmass causes a higher shear stress to be generated in the reinforcing element and joint intersection leading to an overall strength decrease (Ferrero, 1995). Dight (1982) and Schubert (1984) also indicated the strong influence of the rockmass on the deformability and overall resistance to shear in the reinforcing element.

The main focus in the design of the testing facility was to best represent what occurs in an actual underground excavation. The interaction between the rockmass and the reinforcing element has a major influence on the stability of the underground opening; hence the design steered away from the usual guillotine testing facility. Since the focus was on replicating the actual underground mining environment and the role of the reinforcing element to resist shear deformation, the design of the testing facility included installation of the reinforcing elements in a block or bedded rockmass by conventional means.

The laboratory testing facility is located at the School of Mining Engineering in the University of New South Wales and provides a controlled environment for
future evaluation of numerous current and future installed reinforcing products or concepts under shear loading, as part of an ACARP funded project.

An Avery-Denison compressive testing was modified and became the centre piece of the new shear testing facility. The Avery-Denison testing machine, type 7112CCG is capable of loads up to 3600 kN and is shown in Figure 35.

To replicate a similar environment that the reinforcing elements are subject to in an actual underground mining environment, the design of the shear test facility had to allow for:

- Determining the shear displacement along an anticipated plane of weakness and final deformation
- Inducing loading at right angles
- Inducing loading at acute angles
- Determining the load distribution along the reinforcing element as a result of the shear load and displacement
- Determining the load distribution around the reinforcing element (within the concrete/rockmass)

In order to achieve these, the design allowed for two concrete specimens to be cast and subsequently coupled together through installation of a reinforcing element recreating a rockmass with known material properties and uniform strength with a single dominant joint structure, which the shear loads were applied across. The purpose of recreating a rockmass through casting concrete was to control the rockmass material properties and accurately measure the mechanical and material properties of the concrete. The joint plane that was created with the use of concrete can be controlled and altered through the location, size, roughness and other properties of the joint.

The decision to develop and construct a single shear plane facility compared to a double embedment type arrangement was due to the Avery-Denison layout and previous design iterations. There are current shear test facilities that utilise the double shear method and these facilities minimise the potential rotation of the testing blocks, which can be common in a single shear plane facility. This rotation had to be monitored and minimised during the testing program.

The concrete specimens were cast in a steel RHS casing of 8mm thickness. The intention was to fully enclose the concrete specimens to prevent splitting of the specimen, which was an issue in earlier experiments by Hass (1976).

An initial schematic diagram of the shear test facility is shown in Figure 36 (Hartman & Hebblewhite, 2003).
Figure 36 – Schematic diagram of the initial shear test facility (Hartman & Hebblewhite, 2003)
A concern with the initial design was the high structural requirements to resist any movement in the larger of the two blocks especially with the formation of the tensile forces generated in the floor bolts used to fix the reactive structure to the floor. The strongest point on the Avery-Denison testing machine is in the top platen which was comprised of an adjustable spherical seat that can be adjusted up and down. It was and designed to resist the pressures dissipated by the lower piston through the entire structural frame.

Alternate forms of shear testing common within the field of civil engineering were considered including the use of large, rigid reactive frames that are constructed and configured on a laboratory floor. This was considered not practicable for the project requirements. This method was not suitable when incorporating a reinforcing element into the system and does not provide sufficient flexibility to analyse a variety of factors that influence the performance of reinforcing elements under shear loading conditions.

The design was then altered to take advantage of the reactive capacity of the Avery-Denison structural frame, off-setting the smaller and larger specimen blocks as shown schematically in Figure 37.
This design change induces the shear force directly through the shear plane and therefore on the reinforcing element. This helps to achieve static equilibrium and further structural eccentricity throughout the entire shear testing facility. An upward pressure force from the bottom piston can be isolated on the smaller specimen and the reactive force on the larger specimen. This configuration gives the smaller specimen a surface to displace and further prevents rotation of the smaller specimen, which was representative of the arrangement in the field. The initial off-set distance between the two specimens was approximately 50mm.

An issue considered was the compliance of the spherical seat that was capable of being retractable through an electric motor and gear drive assembly. The spherical seat was designed to ensure that a direct reactive force was applied though the centre of the core specimen despite any minor symmetric irregularities. The spherical seat was configured in such a way that it can pivot within a restricted range and this was a major issue as the objective was to prevent any rotation of the test specimens. An example of the spherical seat that has been un-bolted and exposed can be seen in Figure 38. This illustrates
the dome plate that rotates on the concave screw thread in the top frame of the Avery-Denison testing machine.

Figure 38 – Orientation of the spherical set in the Avery-Denison testing machine

There were two main options in removing the rotating capabilities of the spherical seat including inserting a steel ring around the convex dome and removing the gap that was present to allow rotation, or fixing a solid surface horizontally underneath the spherical seat to a fixed point on or outside the Avery-Denison testing machine preventing the ability to rotate.
There was evidence that the installation of a steel ring has worked effectively in previous arrangements. A major issue was ensuring contact was maintained directly through the centre of the spherical seat and the above concave dome screw thread. This could otherwise lead to the Avery-Denison being operated out of the designed capacity limits. Due to the high stress concentrations on this section of the Avery-Denison testing machine the steel ring was not incorporated into the design of the shear testing facility.

The use of a fixed steel beam to provide a horizontal bearing surface for the spherical seat to rest directly on the beam would help to prevent sample rotation. A schematic of this set-up is shown in Figure 39.

![Figure 39 – Schematic of the orientation of the samples prior to testing](image)

A weakness of this design would be the amount of deflection of the steel beam which can lead to a slight rotation of the spherical seat.

The final design of the shear testing facility is shown in Figure 40. The spherical seat has been removed and retracted the void of the machine. The main reason for this was to increase the maximum clearance between the top of the piston and bottom of the top supportive frame of the Avery-Denison testing. This allowed for other components of the shear testing facility including the bottom plate and beams, size of the two specimens and top reactive beam. Completely
removing the spherical seat would provide an extra 80mm to adjust the other components accordingly.

The Avery-Denison testing machine is located over a 440 x 550 mm concrete beam in the underlying floor structure. This provides a solid footing for the Avery-Denison testing machine during the testing process.

### 3.2. The Avery-Denison testing machine

The Avery-Denison testing machine is a hydraulic universal press that has a capacity of 3600 kN. The Avery-Denison testing machine is comprised of an indicator/control console containing the pumping unit and valve gear, a dial indicator and electrical controls. It is connected up to the hydraulic piston via flexible pipes and cables. The main piston is located within the walls of a cylinder that has hydraulic fluid fed via the top and bottom of the piston. The piston is contained by hydraulic fluid within the cylinder walls to provide constant lubrication to the piston and minimise friction at high loads.

Load is applied by the main ram and measured using the dial. Both rams work without packing and the proportional ram is rotated continuously to eliminate friction. This hydraulic method of load transmission is sensitive and ensures that initial accuracy is maintained.

The force from the proportional ram is transmitted through an intermediate lever to a capacity-change lever provided with an appropriate fulcra, which are selected by a handwheel and adjustment of the pendulum weights at the rear of the dial indicator. A reduced force from the capacity-change lever displaces the pendulum and, through a rack and pinion, rotates the pointer.

There are two main handwheel controls, one for increasing and the other for reducing the load. The loading valve is designed so that at any setting the rate of oil flow is uniform. The unloading valve is similar and, in combination, the two
give all the control needed for applying an incremental load, for applying a load quickly, for holding loads steady and for removing loads. There is a master relief valve on the side of the indicator unit for quick removal of the entire load.

For the current research requirements, the chart range was in the ratio of one fifth of the capacity of the Avery and the load indicated by the analogue chart and later with the installation of a pressure transducer.

A schematic drawing of the finalised design of the shear testing facility can be seen in Figure 40.
The test facility incorporates two side benches on either side of the Avery-Denison testing machine, one was used to support the larger specimen and the other to support the top reaction beams. The bench that was to support the larger specimen was raised by 70mm to accommodate the off-set specimens and the rigid solid plate that was to be used to exert the loads to the smaller specimen. The benches are dyna-bolted to the ground and cannot provide any resistance in tension were used as in the initial design.

The lower piston of the Avery-Denison testing machine applies the load to the smaller of the two test specimens in a test. The lower steel block, shown in Figure 40, is comprised of two 150UC30 beams with a length of 520mm welded to a top plate of 20mm thickness and a bottom plate of 10mm thickness. Web stiffeners are located halfway up the two beams where the lower steel block was in contact with the smaller specimen. Inclusion of the web stiffeners provides reinforcement of the beam webs and increases the loading capacity through the truss action.

To ensure the applied load was concentrated along the axis of the shear plane, two narrow steel bars were to be added as shown in Figure 41.
This feature though was discarded as the concentrated load would crush/distort the ends of the steel casing. The incorporation of the lower steel block into the design would tend to ensure a more uniform load under the smaller specimen.

### 3.3. *Finite Element Analysis*

A Finite Element Analysis (FEA) was undertaken in regards to the interaction of the lower steel block and the smaller specimen using the Phase² software program. The plane-strain elastic analysis considered the vertical stresses acting on the smaller specimen and lower steel block.

The interaction between the smaller and larger specimens that creates the shearing joint plane and the resistance through the reinforcing element was assumed to have a zero horizontal displacement. Further, the vertical load was directly taken in shear by the reinforcing element. It was assumed there was a smooth joint interface between the smaller and larger specimen though in
practice the joint has a tendency to open and/or close while both specimens can also deform when load was applied.

With the use of an elastic analysis only, there was a possibility of local yielding at the reinforcing element however the focus of the analysis was the behaviour of the lower steel block and the smaller specimen.

It was assumed that the piston and lower steel block were welded together and therefore no sliding could occur. The piston has side rolling constraints that represent the piston ram on the cylinder walls. The applied load was directly on the base of the piston and the use of side rollers on the piston was applicable due to lubrication and oil present between the piston and the cylinder walls. In the plane-strain analysis, a rectangular-shaped piston was modelled due to the 2-dimensional analysis limitations and therefore no change in shear strain. The piston and lower steel block both were assigned values for Elastic Modulus of 200 GPa and a Poisson’s Ratio of 0.25.

While the smaller specimen consisted of a concrete mass encased within a steel shell the FEA model assumed a single material with an Elastic Modulus of 120 GPa with a Poisson’s Ratio of 0.2.

Quadrilateral elements with eight nodes were used to generate a mesh with a finer mesh in areas where displacements were estimated to vary very rapidly. The mesh used for this analysis is shown in Figure 42.
Results of modelling can be seen in Figure 43 with a concentration of load about the rockbolt.

The grey outline represents an exaggerated deformation path. The scale ranges from 0.0 MPa on the left hand side in the maroon colour to 3.5 MPa on the right.
hand side in a dark blue colour. This linear variation in load in the model confirms the desired uniformly distributed load under the smaller specimen which best replicates the occurrence in the underground mining environment.

3.4. Reaction Beams

The two reaction beams at the top of the shear testing facility were installed in order to counteract the eccentricity of the load point. The beams are 150UC30 type beams. Since there was to be high shear loads generated it was recommended that 10mm plate web stiffeners be added (Leedow, 2004). Dimensions of the 150UC30 beams are provided in Table 2, (AISC, 1999).

<table>
<thead>
<tr>
<th>Designation</th>
<th>Depth of Section</th>
<th>Flange Width</th>
<th>Flange Thickness</th>
<th>Web Thickness</th>
<th>Root Radius</th>
<th>Depth Between Flanges</th>
</tr>
</thead>
<tbody>
<tr>
<td>kg/m</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td>30.0</td>
<td>158</td>
<td>153</td>
<td>9.4</td>
<td>6.6</td>
<td>8.9</td>
<td>139</td>
</tr>
</tbody>
</table>

Table 2 – Dimensions of the 150UC30 beams (AISC, 1999)
The top reaction beams had handles installed at either end to allow easy positioning of the beams over the specimen prior to testing.

Bearing Plates of 20mm thickness were added to counteract the eccentricity and provide a bearing surface area during loading.

### 3.5. **Steel Casings**

Steel casing fabricated from 8mm plate steel were used to confine the concrete specimens as shown in Figure 44.

![Figure 44 – Original RHS steel casings](image)

The steel casings were of 9.5 mm thickness. The smaller specimen steel casing had the dimension 305 mm x 305 mm x 250 mm, where the larger specimen steel casing had the dimension 305 mm x 305 mm x 1000 mm.
Lifting hooks were added on both sides of the casing to manoeuvre the casings. The casings were later split along their length to aid in removal of the embedded concrete after testing. Flanges were welded to both sections, reinforced and bolted together as shown in Figure 45. Flanges were also welded at one end of both the large and small casings. These were to allow the casings to be bolted together when drilling and installing the rockbolts. The flanges were positioned so that the casings were offset to each other by 45mm as illustrated in Figure 45 with the fully assembled test sample.

An additional four steel casings (Mark II) were fabricated from using “u” sections and joined together by a flange as shown in Figure 46. This enabled easy removal of the concrete specimen after testing was complete. All steel casings were coated with a corrosion resistant paint.
3.6. **Pressure Transducer**

A pressure transducer was installed to monitor the pressure on the piston. A panel meter calibrated in kN provided a real time display of the load during a test.

The range of the pressure transducer was 0-20MPa (200 bar). This was less than the capacity range of the Avery-Denison testing machine of 350 bar (5000 psi) A 200 bar pressure transducer allowed for a maximum load of 720kN to be applied.

A PM4 load cell monitor and panel meter was used for the digital pressure readouts. This monitor was a high precision load cell/strain cell monitor with a sampling rate of up to 100 samples per second. The load cell monitor also provided an output signal that could be connected to data logger. The signal had a calibrated output of 0-10 Volt.
3.7. **LVDT Transducer**

A series of Linear Variable Differential Transformers (LVDTs) were used in the tests to measure displacement.

Solartron BS 25 MA LVDTs were used in the test with the following specifications:
- MACH 1 series
- ±25 mm stroke displacement LVDT transducer
- 5Khz rated to 150° C
- axial cable outlet
- Ø 6.35mm guided carrier and core
- M4 x 0.7 thread with fitted with a BICM unit
- springs to retract inner core

Technical and specification sheets of the LVDT transducers can be found in the appendix A4.

3.8. **Calibration**

The Avery-Denison testing machine was recalibrated by Precision Calibration Services (NATA Accredited Laboratory Number: 1710) prior to testing, in accordance with Australian Standard 2193 (2002) *Calibration and classification of force measuring systems*. Recalibration is recommended prior to 3rd September 2006.

The digital indicator was calibrated to 700kN at 0.1kN graduations with a readability of 0.1kN.
The dial gauge on the Avery-Denison testing machine was calibrated to a capacity of 720kN at 5kN graduation and 1kN readability. After the initial applied force of 100kN, the grade Class was ‘B’ with the analogue scale then calibrated with an ‘A’ Class rating up to 720kN in accordance with AS 2193.

### 3.9. Analysis of Test Facility

#### 3.9.1. Free Body Diagrams

To facilitate an analysis of the forces generated on the different components of the shear testing facility during testing, the test facility was broken down into the following components:

- Piston and lower steel block
- Smaller specimen test block
- Larger specimen test block
- Top reaction beams
- Vertical support beams
- Support benches

The piston and lower steel block are the active members that apply the load to the smaller specimen. Working together, the objective was to replicate a uniformly distributed load under the smaller specimen as would be found in the underground mining environment. Since the piston is unguided on the Avery-Denison testing machine it was believed that there may be minor frictional pressure from the piston onto the walls of the cylinder caused by rotational or off-centre movement that would be initiated during shear testing.

A free body diagram of the predicted forces exerted onto the piston and lower steel block is shown in Figure 47.
A sketch of the forces exerted onto the smaller specimen can be seen in the free body diagram shown in Figure 48. There are some rotational forces that were exerted onto the larger specimen at the top right-hand section of the smaller specimen.
Interaction between the smaller specimen and the larger specimen was subject to additional reactive forces in order to keep the larger specimen as fixed and rigid as possible. The free body diagram of the forces exerted onto the larger specimen can be seen in Figure 49.

![Free body diagram of the forces applied onto the larger specimen](image)

Figure 49 – Free body diagram of the predicted forces applied onto the larger specimen

The top reaction beams are subjected to forces from the larger specimen through the bearing plate that was located between the top reaction beams and the larger specimen. The top reaction beams are directly in contact with the top of the Avery-Denison testing machine and the free body diagram of these beams is shown in Figure 50.

![Free body diagram of the top reaction beams](image)

Figure 50 – Free body diagram of the top reaction beams
The top of the Avery-Denison testing machine was predicted to equally exert a reactive force from the top reaction beams and supported by the four column supports on the Avery-Denison testing machine to resist these forces. From the top reaction beams, a force was exerted onto the vertical support beams that are in contact with the original bench. These vertical beams are designed similar to the top support beams and have a free body diagram as shown in Figure 51.

3.9.2. Load Calculations

Calculations were undertaken for applied loads up to 600kN. This was the initial load limit for the shear testing facility. To calculate the forces present in the larger specimen, Figure 52 was used below to examine these forces (Leedow, 2004).
Figure 52 – Forces within the larger specimen (from Leedow, 2004)

\[ \sum M_C = 0 \]
\[ \sum M_C = 600 \text{kN} \times 0.475 - F_B \times 0.350 + F_D \times 0.450 \]
\[ 600 \text{kN} \times 0.475 - F_B \times 0.350 + F_D \times 0.450 = 0 \]

*Assume \( F_D = 0 \) (Tension not resisted)*

\[ F_B = 815 \text{kN} \]
\[ M_B = 75 \text{kNm} \]
\[ F_C = 215 \text{kN} \]

A summary of the forces acting on the larger specimen can be seen in Figure 53 (Leedow, 2004).
The next main interaction in the shear testing facility was the top reaction beams that are in contact with the larger specimen, top of the Avery-Denison testing machine and the vertical support beams. The free body diagram of the beams can be seen in Figure 54 with the calculation of the forces on the beam shown in Figure 55 (Leedow, 2004).

$$F_B = F_D$$
\[ \Sigma M_A = 0 \]
\[ \Sigma M_A = F_B \times 0.270 + F_D \times (0.270 + 0.375 + 0.125) - 815 \times (0.270 + 0.375) \]
\[ F_B \times 0.270 + F_D \times (0.270 + 0.375 + 0.125) - 815 \times (0.270 + 0.375) = 0 \]
\[ 2F_B = 1010 \text{ kN} \]
\[ F_B = F_D = 505 \text{ kN} \]

\[ F_A = 505 + 505 - 815 \]
\[ F_A = 195 \text{ kN} \]

A summary of the forces acting on the top reaction beams can be seen in Figure 55 (Leedow, 2004).

Figure 55 – Load calculations within the larger specimen (from Leedow, 2004)

600 kN was the maximum applied load used to determine the thickness of the steel plates, casing and beams that were subjected to the applied loads. This maximum applied load was utilised due to previous research and recommendations indicating that conventional 22 mm diameter rockbolts fail under shear loading between 300 kN to 400 kN.
4. Concrete Casting

4.1. Introduction

The importance of the surrounding rockmass is critical in analysing the performance of reinforcing element under a shear load. The testing regime was comprised of concrete to represent the surrounding rockmass representing typical values of rockmass in both strength and mechanical properties.

4.2. Concrete

Concrete was used to represent the surrounding rockmass during testing of the reinforcing elements. The ability for concrete to be made plastic and to gradually harden to form a stone-like material was ideal for this casting.

As part of the project objectives, it was important to carry out appropriate tests on the hardened concrete to determine if there were any changes in the physical properties of the concrete including the compressive strength, static modulus of elasticity and Poisson’s ratio in compression.

During pouring of the concrete into the steel casings, we poured cylindrical test specimens (100mm diameter by 200mm high) for determination of the properties of the concrete as shown in Figure 56.
The compressive tests were undertaken as per the American Society of Testing Materials (ATSM) by the School of Civil and Environmental Engineering at the University of New South Wales.

### 4.3. Property Tests

To determine the compressive strength of the concrete specimens, compressive tests were undertaken in accordance with ATSM Designation: C 39.

The determination of Static Modulus of Elasticity (E) and Poisson’s Ratio (ν) were done in accordance with ATSM Designation: C 469. Young’s Modulus and Poisson’s Ratio of portland cement concrete was determined under longitudinal compression using the chord modulus to define elasticity. For normal weight concrete E ranges from 14 to 41 GPa.
The Static Modulus of Elasticity (E) was calculated by the formula:

\[
E = \frac{\sigma_2 - \sigma_1}{\varepsilon_2 - 0.00005}
\]  \hspace{1cm} (8)

where:

- \(E\) : Chord modulus of elasticity
- \(\sigma_2\) : Stress corresponding to 40% of the estimated ultimate load or ultimate stress, based upon previously tested specimens in accordance with ASTM Designation: C 39
- \(\sigma_1\) : Stress corresponding to a longitudinal strain of 0.000050
- \(\varepsilon_2\) : Longitudinal strain corresponding to the \(\sigma_2\) stress
- \(\varepsilon_1\) : Strain 0.000050 and, therefore, does not appear in the formula for \(E\)

Strain gauges were attached to the standard cylindrical specimens; the specimens were loaded and unloaded to 10 kN, primarily to properly seat the strain gauges. The concrete was then loaded at a slow, steady rate and recorded accordingly. During the initial loading procedure no data was recorded, this was required for the seating of the gauges, to observe the performance of the assembly and to note any problems. Longitudinal and transverse strain readings at 25% and 40% of the estimated ultimate compressive load were recorded after the initial loading sequence for the following two loading sequences and this was repeated twice for every sample. The results of the two cycles are averaged and compared to one another after the tests.

Poisson’s Ratio was calculated based on the transverse strains at the mid-height of the specimen. The typical values for Poisson’s ratio for concrete range between 0.15 to 0.25, with the aggregate, moisture content, age of the concrete and compressive strength being contributing factors (Mindess & Young, 1981).
Poisson’s Ratio ($\nu$) was calculated by the formula:

$$
\nu = \frac{(\varepsilon_{t2} - \varepsilon_{t1})}{(\varepsilon_2 - 0.000050)} \quad (9)
$$

where:

- $\varepsilon_{t2} - \varepsilon_{t1}$: The transverse strain divided by 2 at stresses $\sigma_2$ and $\sigma_1$, respectfully
- $\varepsilon_2$: Longitudinal strain corresponding to the $\sigma_2$ stress

### 4.3.1. Stage 1 Concrete Test Results

Testing of the concrete cylindrical specimens was undertaken at the School of Civil and Environmental Engineering at the University of New South Wales as per the American Society of Testing Materials (ATSM) standards.

Dimensions of the cylinders were recorded prior to testing. All specimens were tested using a Satec servo-controlled compressive testing machine as shown in Figure 57.
The specifications of the concrete had a 28 day strength of 60 MPa. Testing occurred when the cylinders had cured for 52 days.

Results of the concrete cylinder compression test are summarised in Table 3.

Table 3 – Results from Stage 1 concrete cylinder compression test

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
<th>P4</th>
<th>P5</th>
<th>P6</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Diameter</td>
<td>mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Load</td>
<td>kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength</td>
<td>MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

90

The strength of the concrete exceeded 60 MPa with an average strength of 65.9 MPa.

Figure 58 shows graphically the compressive strength of specimen P1 with respect to the ram displacement on the compressive testing machine.
Figure 58 – Compressive strength of specimen P1 with respect to ram displacement on the compressive testing machine.

Figure 59 shows an “hourglass” behaviour that is common to these type of cylindrical specimens after exceeding the compressive strength of the concrete.

Figure 59 – Failed concrete cylindrical specimen after compressive testing
To determine the Static Modulus of Elasticity and Poisson’s Ratio of the concrete in accordance with ATSM Designation: C 469, both ends of the specimen were capped with contact pads as shown in Figure 60.

![Figure 60 – Figure of longitudinal strain gauge positioned on the concrete test specimens](image)

Static Modulus of Elasticity results are summarised in Table 4.

**Table 4 – Static Modulus of Elasticity of Stage 1 tests**

<table>
<thead>
<tr>
<th></th>
<th>2nd Reading</th>
<th>3rd Reading</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen P4</td>
<td>40.6 GPa</td>
<td>40.7 GPa</td>
<td>40.7 GPa</td>
</tr>
<tr>
<td>Specimen P5</td>
<td>36.9 GPa</td>
<td>37.2 GPa</td>
<td>37.1 GPa</td>
</tr>
<tr>
<td>Specimen P6</td>
<td>N/A GPa</td>
<td>37.4 GPa</td>
<td>37.4 GPa</td>
</tr>
<tr>
<td>Average</td>
<td>38.8 GPa</td>
<td>38.4 GPa</td>
<td>38.4 GPa</td>
</tr>
</tbody>
</table>
The Static Modulus of Elasticity was loaded in three cycles as shown in Figures 61 to 63. Loading of the first cycle was not shown as it allowed for the seating of the gauges, to observe any problems with the instrumentation and to remove any hysteresis in the loading system. The Static Modulus of Elasticity was determined from the second and third loading cycles.

Figure 61 – Static chord modulus of elasticity for specimen P4

Figure 62 – Static chord modulus of elasticity for specimen P5
In order to obtain values of Poisson’s ratio, a transverse strain gauge was attached around the centre of the cylindrical concrete specimen as shown in Figure 64. This was used to record the transverse strain as well as the longitudinal strain.
Results of Poisson’s Ratio are shown in Table 5.

Table 5 – Poisson’s Ratio Stage 1 test results

<table>
<thead>
<tr>
<th></th>
<th>Poisson’s Ratio 1</th>
<th>Poisson’s Ratio 2</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen P7</td>
<td>0.12</td>
<td>0.11</td>
<td>0.12</td>
</tr>
<tr>
<td>Specimen P8</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Specimen P7 displayed relatively low Poisson’s Ratio levels due to the high strength concrete properties.

Specimen P8 failed during the initial loading cycle as the servo controlled compressive machine failed to reduce the applied load onto the specimen and exceeded the compressive strength of the concrete cylinder.

### 4.3.2. Stage 2 Concrete Test Results

Concrete tests were undertaken at 28 and 31 days after the pouring of the concrete. There were 12 cylindrical specimens cast for one 14-day compressive strength check and five tests at 28 days, five tests at 31 days and one spare.

The 14-day UCS strength was 48.9 MPa with an estimated Modulus of 31.1 GPa. The 14-day UCS strength was relatively high compared to the properties of the concrete having a 28-day UCS strength of 40 MPa. This deviation was sought to be the poor quality, high temperature, low workability of the concrete that was transported on site and also a defective cylindrical specimen.
4.3.2.1. 28 Day Test

The 28-day UCS tests on the concrete were undertaken on two concrete specimens and the results from the concrete cylinder compression test can be seen in Table 6.

Table 6 – Results from Stage 2, 28 day concrete cylinder compression test

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>CS2-1</th>
<th>CS2-2</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Diameter</td>
<td>mm</td>
<td>99.9</td>
<td>99.8</td>
</tr>
<tr>
<td>Maximum Load</td>
<td>kN</td>
<td>355.0</td>
<td>348.0</td>
</tr>
<tr>
<td>Strength</td>
<td>MPa</td>
<td>45.3</td>
<td>44.5</td>
</tr>
</tbody>
</table>

The strength of the concrete averages 44.9 MPa in 28 days of curing, which was a little higher than the required 40 MPa that was the initial requirement for the Stage 1 tests.

![Figure 65](image)

Figure 65 – 28 day compressive strength of specimen CS2-1 with respect to ram displacement on the compressive testing machine

Figure 66 shows the failure of specimen CS2-1.
The Static Modulus of Elasticity was determined for the 28-day concrete specimens CS2-3 and CS2-4 and Poisson’s Ratio calculated for specimen CS2-5.

Static Modulus of Elasticity results of each of the Stage 2, 28-day cylinder test specimens we recorded and shown in Table 7.

Table 7 - Static Modulus of Elasticity results from the Stage 2, 28-day concrete test specimens

<table>
<thead>
<tr>
<th></th>
<th>1st Reading</th>
<th>2nd Reading</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Specimen CS2-3</strong></td>
<td>32.5</td>
<td>33.1</td>
<td>32.8</td>
</tr>
<tr>
<td><strong>Specimen CS2-4</strong></td>
<td>32.1</td>
<td>31.9</td>
<td>32.0</td>
</tr>
<tr>
<td><strong>Specimen CS2-5</strong></td>
<td>N/A</td>
<td>31.6</td>
<td>31.6</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>32.3</strong></td>
<td><strong>32.2</strong></td>
<td><strong>32.3</strong></td>
</tr>
</tbody>
</table>
Figure 67 shows the results in determining the Static Chord Modulus of Elasticity of specimen CS2-3. Hysteresis was evident in the first reading compared to the second and third readings. The second and third readings overlap and it was these readings that were used to calculate the Modulus of Elasticity of the concrete specimens.

![Graph showing stress-strain relationship with readings](image)

Figure 67 – Static Modulus of Elasticity results of specimen CS2-3 at 28 days

Sample CS2-5 had an average Poisson’s Ratio of 0.14 as shown in Table 8.

Table 8 – Poisson’s Ratio Stage 2, 28-day concrete test results

<table>
<thead>
<tr>
<th>Sample</th>
<th>Poisson’s Ratio 1</th>
<th>Poisson’s Ratio 2</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS2-5</td>
<td>0.15</td>
<td>0.13</td>
<td>0.14</td>
</tr>
</tbody>
</table>

4.3.2.2. 31 Day Test

UCS tests were undertaken after 31 days on two concrete specimens and the results from the concrete cylinder compression test can be seen in Table 9.
Table 9 – Results from Stage 2, 31-day concrete cylinder compression test

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>CS2-3</th>
<th>CS2-4</th>
<th>CS2-6</th>
<th>CS2-7</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Diameter</td>
<td>mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>99.8</td>
<td>100.0</td>
<td>100.1</td>
<td>99.9</td>
<td>100.0</td>
</tr>
<tr>
<td>Maximum Load</td>
<td>kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>378.0</td>
<td>376.0</td>
<td>371.5</td>
<td>347.0</td>
<td>368.1</td>
</tr>
<tr>
<td>Strength</td>
<td>MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>48.3</td>
<td>47.9</td>
<td>47.2</td>
<td>44.3</td>
<td>46.9</td>
</tr>
</tbody>
</table>

UCS of the 31-day concrete specimens attained higher values than the 28-day tests and averaged 46.9 MPa.

![Graph showing stress vs. displacement for specimen CS2-6](image)

Figure 68 – 31-day compressive strength of specimen CS2-6 with respect to ram displacement on the compressive testing machine

The Static Modulus of Elasticity was determined for the 31-day concrete specimens CS2-8 and CS2-9 and Poisson’s Ratio calculated for specimen CS2-10.

Static Modulus of Elasticity results of each of the Stage 2, 31-day cylinder test specimens we recorded and shown in Table 10.
Table 10 – Static Modulus of Elasticity results from the Stage 2, 32-day concrete test specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>1st Reading</th>
<th>2nd Reading</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS2-8</td>
<td>34.8</td>
<td>34.8</td>
<td>34.8</td>
</tr>
<tr>
<td>CS2-9</td>
<td>33.9</td>
<td>33.8</td>
<td>33.9</td>
</tr>
<tr>
<td>CS2-10</td>
<td>N/A</td>
<td>35.0</td>
<td>35.0</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>34.4</strong></td>
<td><strong>34.5</strong></td>
<td><strong>34.5</strong></td>
</tr>
</tbody>
</table>

Sample CS2-10 had an average Poisson’s Ratio of 0.13 as shown in Table 11.

Table 11 – Poisson’s Ratio Stage 2, 31 day concrete test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Poisson’s Ratio 1</th>
<th>Poisson’s Ratio 2</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS2-10</td>
<td>0.13</td>
<td>0.13</td>
<td>0.13</td>
</tr>
</tbody>
</table>

4.3.3. Stage 3 Concrete Test Results

Concrete testing was undertaken at 34 and 41 days after the pouring of the concrete. There were 12 cylindrical specimens poured, one for a 14-day compressive strength check, five tests at 34 days and five tests at 41 days with one spare. The 14-day UCS strength was 57.2 MPa.

4.3.3.1. 34 Day Test

The 34-day UCS tests were undertaken on all five of the concrete specimens and the results from the concrete cylinder compression test can be seen in Table 12.
Table 12 – Results from Stage 3, 34 day concrete cylinder compression test

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>CS3-1</th>
<th>CS3-2</th>
<th>CS3-3</th>
<th>CS3-4</th>
<th>CS3-5</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Diameter mm</td>
<td>100.0</td>
<td>99.9</td>
<td>99.8</td>
<td>100.0</td>
<td>100.4</td>
<td><strong>100.0</strong></td>
</tr>
<tr>
<td>Maximum Load kN</td>
<td>N/A</td>
<td>548</td>
<td>566</td>
<td>521</td>
<td>525</td>
<td><strong>540</strong></td>
</tr>
<tr>
<td>Strength MPa</td>
<td>N/A</td>
<td>69.9</td>
<td>72.3</td>
<td>66.4</td>
<td>66.3</td>
<td><strong>68.7</strong></td>
</tr>
</tbody>
</table>

The UCS strength of the concrete reached a maximum of 72.3 MPa and the four recorded concrete specimens averaged 68.7 MPa. The first specimen failed early with no accurate reading taken. The load displacement curve of specimen CS3-3 can be seen in Figure 69.

![Ram displacement (mm) vs Stress (MPa)](image)

Figure 69 – 34-day compressive strength of specimen CS3-3 with respect to ram displacement on the compressive testing machine

Static Modulus of Elasticity results for each of the Stage 3, 34-day cylinder test specimens averaged 32.2 GPa and are shown in Table 13.
Table 13 – Static Modulus of Elasticity results from the Stage 3, 34-day concrete test specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>1st Reading</th>
<th>2nd Reading</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS3-4</td>
<td>31.9</td>
<td>32.1</td>
<td>32.0</td>
</tr>
<tr>
<td>CS3-5</td>
<td>32.3</td>
<td>32.4</td>
<td>32.4</td>
</tr>
<tr>
<td>Average</td>
<td>32.1</td>
<td>32.3</td>
<td>32.2</td>
</tr>
</tbody>
</table>

Sample CS3-5 had an average Poisson’s Ratio of 0.13 as shown in Table 14.

Table 14 – Poisson’s Ratio Stage 1 test results from the Stage 3, 34 day concrete test specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Poisson’s Ratio 1</th>
<th>Poisson’s Ratio 2</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS3-5</td>
<td>0.13</td>
<td>0.13</td>
<td>0.13</td>
</tr>
</tbody>
</table>

4.3.3.2. 41 Day Test

Tests were undertaken 41-days after pouring that included five compression, two Static Modulus of Elasticity and one Poisson’s Ratio test. The 41 days coincided with the testing of the strain-gauged reinforced samples. The compressive test results can be seen in Table 15 and the Static Modulus of Elasticity results are shown in Table 16. The five concrete specimens had an average UCS strength of 70.5 MPa and a Static Modulus of Elasticity of 32.7 GPa.
Table 15 – Results from Stage 3, 41 day concrete cylinder compression test

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>CS3-6</th>
<th>CS3-7</th>
<th>CS3-8</th>
<th>CS3-9</th>
<th>CS3-10</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>100.0</td>
<td>99.9</td>
<td>99.6</td>
<td>99.5</td>
<td>100.0</td>
<td>99.8</td>
<td></td>
</tr>
<tr>
<td>Maximum Load</td>
<td>kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>515</td>
<td>561</td>
<td>566</td>
<td>555</td>
<td>561</td>
<td>553</td>
<td></td>
</tr>
<tr>
<td>Strength</td>
<td>MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>65.6</td>
<td>71.6</td>
<td>72.6</td>
<td>71.4</td>
<td>71.5</td>
<td>70.5</td>
<td></td>
</tr>
</tbody>
</table>

Table 16 – Static Modulus of Elasticity results from the Stage 3, 41 day concrete test specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>1st Reading</th>
<th>2nd Reading</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS3-8</td>
<td>32.2</td>
<td>32.6</td>
<td>32.4</td>
</tr>
<tr>
<td>CS3-9</td>
<td>32.8</td>
<td>33.0</td>
<td>32.9</td>
</tr>
<tr>
<td>Average</td>
<td>32.6</td>
<td>32.8</td>
<td>32.7</td>
</tr>
</tbody>
</table>

Sample CS3-10 had an average Poisson’s Ratio of 0.13 as shown in Table 17.

Table 17 – Poisson’s Ratio Stage 1 test results from the Stage 3, 41 day concrete test specimens

<table>
<thead>
<tr>
<th></th>
<th>Poisson’s Ratio 1</th>
<th>Poisson’s Ratio 2</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS3-10</td>
<td>0.13</td>
<td>0.13</td>
<td>0.13</td>
</tr>
</tbody>
</table>

4.4. **Summary**

In summary, the strength of the concrete used to simulate the surrounding rockmass during the testing of the elements ranged from 44.3 MPa to 72.6 MPa. The Static Modulus of Elasticity averaged 32.7 GPa to 38.4 GPa with Poisson’s Ratio resulting in a consistent 0.12 to 0.13. Table 18 shows a summary of the concrete properties in each of the stages of testing. Stage 2
and Stage 3 results are based on the set of tests that were of the longest curing duration.

Table 18 – Summary of concrete property tests

<table>
<thead>
<tr>
<th></th>
<th>Stage 1</th>
<th>Stage 2</th>
<th>Stage 3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strength</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average MPa</td>
<td>65.9</td>
<td>46.9</td>
<td>70.5</td>
</tr>
<tr>
<td>Maximum MPa</td>
<td>71.2</td>
<td>48.3</td>
<td>72.6</td>
</tr>
<tr>
<td>Minimum MPa</td>
<td>62.3</td>
<td>44.3</td>
<td>65.6</td>
</tr>
<tr>
<td><strong>Modulus of Elasticity</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average GPa</td>
<td>38.4</td>
<td>34.5</td>
<td>32.7</td>
</tr>
<tr>
<td>Maximum GPa</td>
<td>40.7</td>
<td>35.0</td>
<td>32.2</td>
</tr>
<tr>
<td>Minimum GPa</td>
<td>36.9</td>
<td>33.8</td>
<td>33.0</td>
</tr>
<tr>
<td><strong>Poisson’s Ratio</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>0.12</td>
<td>0.13</td>
<td>0.13</td>
</tr>
<tr>
<td>Maximum</td>
<td>0.12</td>
<td>0.13</td>
<td>0.13</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.11</td>
<td>0.13</td>
<td>0.13</td>
</tr>
</tbody>
</table>
5. Stage 1 Tests

During commissioning of the shear test facility, two series of tests were undertaken in order to determine the rigidity, functionality and performance of the test facility. These initial tests were also undertaken to determine the overall logistics and procedures associated with determining the shear performance of the reinforcing element. Minor modifications to the facility, systems and logistics were subsequently made.

5.1. Concrete casting

Concrete was used to represent the surrounding rockmass in which the reinforcing element was later installed.

Prior to pouring of the concrete, the steel casings needed to be well prepared and arranged to prevent any irregularities or impurities in the concrete mixture. Since the steel casings were to be reused, it was necessary that they could be removed from the concrete sample with minor difficulties. The ease of removing these casings also increased the integrity of the concrete sample that can be further analysed and importantly reduced the physical and manual handling effort requirements to remove the concrete sample.

To ensure the integrity of the concrete sample and increase the ease of removing the sample from the steel casing, a mould release agent was placed on the inside of the casings prior to pouring of the concrete. Shell Moulding Oil P5 was selected for this purpose. This was an oil based additive containing mould release fluid and used extensively for poured concrete structures with plywood, hardboard or steel shuttering.

Two coatings of the mould release oil were applied to the new steel casings prior to pouring of the concrete. The steel casings were placed on their ends
with the faces that are to be in contact during the shearing process laid flat on a coated section of form plywood. Having the faces of the concrete that are to be in contact during the shearing test cast flat on the plywood created a smooth concrete surface that simulates a smooth and consistent joint surface throughout the testing program. As mentioned in the literature review, the roughness of the joint surface during the process of shearing along a plane has considerable influence on the overall shear performance of the rockmass and controlling the consistency of this parameter was essential.

In order to create a smooth joint surface for the two blocks to shear along, the steel casings were initially going to be raised 2 to 5 mm off the ground by propping each corner of the steel casing. The main purpose being to create a significant concrete lip away from the steel casing that would prevent any chance of the steel casings, both the large and small specimen, coming into contact during the shearing process. This idea was rejected with strong belief that that concrete during the hardening and curing process will purposely develop this surface lip and not require rising of the steel casings, which may also cause a flow and loss of concrete through this void. The steel casings created the required concrete lip without having to raise the steel casings as initially thought due to the casings being slightly raised during the curing process.

A concrete strength of 40 to 45 MPa was selected to simulate the properties of the surrounding rockmass. Common rock types associated with coal strata are rated as medium to strong with a Uniaxial Compressive Strength (UCS) ranging between 25 and 50 MPa. Examples in this classification include claystone, coal, concrete, schist, shale and siltstone. Peng (1998) these common rock types associated with coal strata as being, in order of decreasing strength, sandstone, limestone, sandy shale, shale, clayey shale and clay.

The final composition of the trial concrete was high strength 60 MPa concrete with 20 mm size aggregate and a nominal slump of 120 mm.
Casting of the concrete was undertaken on-site in the laboratory at the School of Mining Engineering. Some minor issues arose in regards to the manoeuvrability and access of the ready-mixed concrete truck. A risk assessment was undertaken prior to delivery of the concrete and can be found in the appendix A6.

The concrete was poured directly into the steel casings. Once the concrete filled a third of the volume of the casing, an agitator was used to remove any air pockets. Pouring continued until the casing was filled. Nine cylindrical test specimens were also poured for determination of concrete strength properties.

An example of the two steel segments used for a test block is shown in Figure 70 and Figure 71.

![Curing of the Stage 1 large concrete specimen](image_url)
5.2. Reinforcing element

The reinforcing element used throughout the testing program was a Jennmar JBX high tensile roofbolt shown in Figure 72. The JBX is a common reinforcing element that is installed in most Australian underground coal mines and constitute for approximately 80,000 rockbolts per month in New South Wales. There are approximately another 60,000 rockbolts with a higher rib profile (such as the JX rockbolts) and 40,000 mild steel rockbolts used mostly for rib support and for conveyor belt and monorail hangers.

The JBX roofbolt was derived from a new high performance steel that has higher toughness and ductility compared to conventional high strength rockbolt
The JBX roofbolt was rolled from an accelerated cooled microalloyed steel designed for mining rockbolt applications. It is now commonly used in a majority of underground coal mines throughout Australia. Jennmar Australia produces around 120,000 JBX rockbolts per month in New South Wales, Australia. A summary of the mechanical and physical properties of the JBX roofbolt is shown in Table 19:

Table 19 – Summary of mechanical and physical properties for Jennmar JBX Roofbolt (after Jennmar, 2004)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical Yield Strength</td>
<td>240kN</td>
</tr>
<tr>
<td>Typical Ultimate Tensile Strength</td>
<td>335kN</td>
</tr>
<tr>
<td>Typical Elongation to Fracture</td>
<td>19-20%</td>
</tr>
<tr>
<td>Thread</td>
<td>24mm cold rolled, 150mm length</td>
</tr>
<tr>
<td></td>
<td>I.S.O. M24, 3 Pitch</td>
</tr>
<tr>
<td>Nut</td>
<td>Hot Forged</td>
</tr>
</tbody>
</table>

The technical and information sheets of both the Jennmar JBX High Tensile Roofbolt and the HSAC840 rod and bar steels that the JBX roofbolt was rolled from are contained in the appendix A3.

The JBX roof bolt used in the test program was 1.2 m in length, 21.6 mm bar core diameter, 23 mm bar diameter across the ribs, steel area of 366 mm$^2$ and consists of a dome ball, washer and resin breakout nut.

### 5.3. Encapsulation medium

The encapsulating medium be used was Minova Lokset resin capsules which was a polyester resin anchor used to anchor reinforcing elements. The Lokset Resin Capsule consists of a reinforced, thixotropic polyester resin mastic in one compartment and an organic peroxide catalyst separated by a physical barrier in the other. Rotation of the bolt during installation ruptures the capsule, shreds
the skin and mixes the two components causing a chemical reaction and transforming the resin mastic into a solid anchor (Minova, 2004).

In order to apply a pre-tension onto the reinforcing element it was necessary to point anchor the far end of the element to the back of the borehole. A two-stage resin capsule was used. This provided sudden initial point anchorage at the far end of the borehole and a pre-tension to be applied. A second slower setting resin then fully encapsulated and locked in the pre-tension throughout the remaining length of the reinforcing element.

A summary of the resin encapsulating medium at an age of 24 hours can be seen in Table 20:

Table 20 – Summary of mechanical and physical properties for Minova Lokset Resin (after Minova, 2004)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial compressive strength</td>
<td>&gt; 60 MPa</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>&gt; 6.5 GPa</td>
</tr>
<tr>
<td>Push out force²</td>
<td>&gt; 72 kN</td>
</tr>
</tbody>
</table>

The initial preliminary trial tests did not include any pre-tension as the objectives of the initial testing was to investigate the competency of the shear testing facility and establish the procedures and logistics required ready to undertake the actual research testing.

The option of using a mix-and-pour type resin was considered in order to control the application and encapsulation of the resin as it was well documented during installation of reinforcing elements by spinning up short length resin cartridges can cause gloving and failure to attain full encapsulation. Mix-and-pour type arrangements on the other hand are noted to develop ‘air pockets’ due to the high viscosity of the resin when it is poured into the borehole (Jennmar, 2005).

² Measured on a 22mm bolt, 50mm encapsulation in 28mm hole, with slow set resin
5.4. Drilling and installation of reinforcing element

Installation of the reinforcing element required the ability to drill an accurate borehole, spin, install and pre-tension the element simulating the procedures that occur in an underground mine environment.

An ARO Series 4000 Roof Bolting Machine as shown in Figure 73 from Hydramatic Engineering at Redhead, New South Wales was used to drill the borehole, install and pre-tension the reinforcing elements.

Figure 73 – Longwall face bolter (ARO brochure, 2004)

The option of installing and commissioning an ARO roof bolting rig within the laboratory at the School of Mining Engineering, the University of New South Wales was considered though dismissed due to the additional services required in the laboratory including an electric motor and hydraulic pumps. The issue of maintenance, service and support also played a role in utilising the ARO roof bolter at Hydramatic Engineering.
An option of casting a length of conduit within the concrete rockmass and later boring out the conduit and applying a rough surface to the borehole was reviewed. This procedure would allow the full testing process from casting to testing to be undertaken directly within the laboratory. This was discarded due to the additional services also required in the laboratory and the move away from the full simulation of installing a reinforcing element such as in the underground working environment.

A table showing the alternative methods of installing the reinforcing element can be seen in the appendix A2.

The small and large concrete samples were bolted together with an off set of 45 mm and arranged ready for the drilling of the borehole as shown in Figure 74.

![Figure 74 – Initial orientation and set-up of concrete specimens prior to drilling](image)

The longwall face bolter was manoeuvred into place with a Franner mobile crane as shown in Figure 75. The bolter was connected to a hydraulic power pack shown in Figure 76.
An 1.2 m length drill steel was used with a 28 mm wing bit to drill the borehole. The final drilled length of the borehole was 1.115 m.

Images of the drilling process can be seen in Figures 77, 78 and 79.
Figure 77 – Drilling of the borehole 1

Figure 78 – Drilling of the borehole 2
Fast set Minova Lokset resin capsules were used to install the rockbolt into the borehole with no pre-tension applied to the rockbolt in the Stage 1 test session as shown in Figures 80 and 81.
5.5. Instrumentation

The shear test facility relies on several forms of instrumentation to record the pressure applied by the Avery-Denison testing machine, displacement of the smaller specimen relative to the larger specimen and stress in the concrete rockmass.

5.5.1. Vibrating wire instrumentation

The instrumentation initially recommended for this project included:
- Vibrating wire (VW) displacement transducers
- Vibrating wire (VW) concrete pressure cells
- Field datalogger

Vibrating wire (or vibrating strip) transducers are commonly used to measure strain, load, pressure, and water level. These sensors produce a frequency signal generated by a vibrating filament. Two measurements are usually made;
the first is the frequency of the vibrating wire. The second is an optimal
temperature measurement that allows compensation of the frequency
measurement.

5.5.1.1. Operation

An increase in pressure on the diaphragm decreases the tension on the
attached wire as shown in Figure 82. This decreases the wire’s resonant
frequency in the same way that loosening a guitar string decreases its
frequency. Thus, the resonant frequency of the vibrating wire sensor decreases
with increasing pressure.

![Figure 82 – Cutaway view showing internal mechanics of vibrating wire sensors](Campbell Scientific, 2003).

Initially the ‘plucking/pickup’ coils are excited with a ‘swept’ frequency. Typically
the datalogger requires 150 ms to sweep through all frequencies. This swept
frequency causes the wire to vibrate at each of the individual frequencies.
Ideally, all frequencies except the resonant frequency of the wire attenuate
within a short time. The wire vibrates with the resonant frequency for a relatively
long time, and as it does so it cuts the lines of flux in the ‘pickup’ coils inducing
the same frequency on the leads to the datalogger.

After waiting for the non-resonant frequencies to attenuate (20 ms), the
datalogger accurately measures how much time it takes to receive a user-
specified number of cycles. Knowing the time and the number of cycles, the
datalogger then computes the square of the frequency (\(=\frac{1}{T^2}\), where T is the
period in milliseconds) (Campbell Scientific, 2003).
5.5.1.2. Unsuitable Application

The main priority of data acquisition for relatively short duration tests on the shear performance of reinforcing elements was that real time dynamic monitoring was required for accurate recording. Since the VW transducers require time for the excitement of a wire to generate a resonate frequency, dynamic real time monitoring was not possible. Even though there was a short time for the non-resonant frequencies to attenuate at 20 ms the datalogger typically takes 150 ms to sweep through all the frequencies for each individual channel. Consequently with several transducers to be recorded, the scanning rate was too slow in this application.

Also strain-gauged bolts cannot be incorporated into a VW interface as each strain gauge must be individually excited, read and transmitted from the datalogger to the computer data acquisition system. The time for the cycle of reading the first strain gauge to the last would adversely impact the integrity of the data, and the correlation between all the strain gauges cannot be analysed effectively as strain-gauged bolts used in this configuration require dynamic real time monitoring.

VW transducers are commonly used for cyclic monitoring of concrete footings/slabs and displacement of major structures. These transducers can be programmed to take readings at certain frequencies (minutes, hours or days etc.) where dynamic real time monitoring is not required.

5.5.1.3. Data Acquisition System (DAQ)

An alternative system of data recording was finally selected. The data acquisition system (DAQ) was a high frequency multi-channel monitoring system where multiple arrays of 64 channels can be measured as shown in Figure 83.
The system was based on a Pentium III 800MHz personal computer and DASYLab software. Sampling rates can be taken at 1000 readings per second with a typical experiment containing between 1000 and 4000 data sets (Hagan and Weckert, 2004).

During a test, the DAQ was used to record the voltage output of the pressure transducer and up to three LVDT transducers.

The DAQ provided data plots of load versus displacement curves (shear and axial) and of loading rates for each test.

5.6. Testing procedure

The first Stage 1 specimen was tested without any data recording. The main purpose of the first preliminary test was to examine the performance of the shear test facility, the interaction between all components of the complete test facility and the response of the test sample subjected to a shear load.
The Stage 1 test specimen was located onto the raised bench and positioned symmetrically within the test facility. The specimen was not positioned horizontal on the raised bench due to protruding welds that was a result of welding the specimen to the flat steel plate during the drilling and installation process as shown in Figure 84. These welds were subsequently removed prior to the commencement of the next test. Figure 85 shows the placement of the reinforced concrete specimen in the shear test facility prior to applying any shear loads.

Figure 84 – Gap between the bearing plate and top reaction beams due to non horizontal placement of reinforced
Figure 85 – Placement of reinforced concrete specimen prior to any shear loads applied

The pressure transducer was reset to zero once there was minor upward movement of the piston. This action accounted for the dead weight of the piston and was adopted throughout the testing program.

Thin sheet packers were placed between the top of the Avery-Denison testing machine and the reaction beams to minimise movement. During the initial application of a load to the smaller specimen, contact was only made at the front bearing plate that transmitted through the top reaction beams and into the top of the Avery-Denison testing machine. There was no interaction between the top reaction beams and the vertical support beams on the opposite bench or between the same top reaction beams and the smaller bearing plate.

A schematic of the interaction contact points during a test can be seen in Figure 86.
Figure 86 – A schematic of the interaction between the specimen and the shear testing facility indicating contact points

5.7. **Stage 1 test results**

5.7.1. Reinforcing element

The first Stage 1 sample (S1-1) was loaded at an undefined to a maximum load of 415 kN. The load was reduced prior to failure of the rockbolt as the shear displacement of 45 mm was reached. This was the maximum displacement
capacity of the shear test facility. A bearing plate was placed between the small specimen and the lower steel block to gain an additional 20 mm of shear displacement within the facility and the load was re-applied. The reinforcing element failed abruptly at a lower load of only 330 kN after reaching a peak load of 390 kN prior to yielding or ‘necking’ of the element. Load values were based on visual observation, as the data acquisition system (DAQ) had not been commenced for this first test.

Testing of Sample S1-2 made use of the DAQ system, which was used to record the applied load. Contact points were observed as in Figure 86 and a similar failure mechanism as in the first test.

A maximum load of 406 kN was recorded during the first loading of Sample S1-2 test before the load was again taken off prior to failure as shown in Figure 87. Once again the maximum shear displacement capacity of 45 mm was reached and a steel plate had to be inserted to increase the shear displacement capacity to 65 mm.

![Figure 87 – Load versus elapsed time during the first pass of the Sample S1-2 prior to failure](image-url)
Load was then re-applied though it appeared that the bolt yielded at a lower load of approximately 39 kN as shown in (Figure 88).

![Figure 88 – Load versus elapsed time during the second loading cycle of Sample S1-2](image)

Failure eventually occurred at 336 kN, which corresponds to the ultimate tensile strength of the JBX roofbolt as indicated by the manufacturer of 335 kN.

Inspection of the failed reinforcing element revealed that the element failed axially after initial shearing. The element was re-aligned towards the vertical and there was evidence of necking prior to failure.

Figure 89 shows the reinforcing element, resin encapsulation and outer concrete rockmass after failure.

Shear loading of the reinforcing element induced crushing of the concrete around the top of the borehole. Few cracks propagated to the steel casing.
Figure 89 – Visual results of the failed reinforcing element, resin encapsulation and the surrounding concrete

The reinforcing element failed abruptly after forming plastic hinges after the element started to yield. It was evident that tensile and compressive zones developed around the element schematically as shown in Figure 90.
The reinforcing elements rotate around to align to the plane of shearing. Yielding was initiated at the plastic hinges which resulted in axial failure of the element as illustrated in Figures 91-94.
Figure 92 – Failure of Sample S1-2 and S1-1

Figure 93 – Failure of Sample S1-2

Figure 94 – Side profile of the failed rockbolt in Sample S1-2
5.7.2. Summary

After undertaking the initial preliminary tests on the performance of reinforcing element subject to shear load, the procedures involved in testing were adjusted.

The applied load in the reinforcing element exceeded 400 kN and did not fail within the designed shear displacement limit of 45 mm. After reloading the element, the element was only capable of sustaining a lower load before failure at 336 kN.

After visual examination of the reinforcing element, it would appear that the element was exposed to both shear and axial forces with ultimate failure occurring axially between the two plastic hinges that were formed during the shearing process. Ultimate failure of the reinforcing element occurred axially as evident by signs of necking and creation of radial and shear lip zones on the failed section. The shear lip zone was an ellipsoid shape compared to a standard symmetric round shape found when failure occurs in pure tension.

The concrete used to simulate the rockmass had an average strength of 65.9 MPa. With such a relatively higher strength rockmass, the extent of crushing around the borehole by the reinforcing element was limited and would be expected to be greater in weaker material. Stronger and stiffer material creates greater confinement and leads to a higher shear stress in the reinforcing element which consequently experiences a reduced global resistance of the reinforced joint plane.

The elastic modulus of the cylindrical concrete specimens was 38.4 GPa, which was relatively high compared to standard concrete.
5.8. Conclusions and Recommendations

The Stage 1 tests were essential in modifying the logistics for the entire testing procedure as well as commissioning of the shear test facility.

Several options were considered within regard to the drilling and installation of the reinforcing element. In the end it was decided to transport the samples to Hydramatic Engineering where they were drilled and the reinforcing elements installed as per current underground mining practice.

Modifications were made to the steel casings to allow easy removal of the concrete once the test had concluded. Lifting hooks were added to the casings to reduce the process of manual handling. A chain block and pulley system was installed to raise the test samples onto the test bench.

The shear testing facility performed as expected with only minor visual deflection during the testing process. The design and construction withstood all calibration and load testing procedures.

The main issue that required a change in design was the shear displacement capacity of 45 mm. The current design required that the system had to be unloaded to allow room for additional packing. The peak loads attained on re-loading was reduced in the second loading cycle and the shear displacement at failure was only marginally higher than the design capacity of 45 mm.

The loading rates were not accurately measured and this may be one influencing parameter in attaining a relatively higher shear displacement at failure.
6. Stage 2 Tests

During the Stage 2 tests, six reinforcing elements were installed into the concrete and then tested in regards to their performance to resist an applied shear load. The objectives of the Stage 2 tests were to examine the effect of pre-tension on the reinforcing element. Three reinforcing elements were to be installed with no pre-tension and three with 40 kN of pre-tension.

6.1. Concrete casting

The concrete was cast at the School of Civil and Environmental Engineering at the University of New South Wales for improved access and assistance. Six steel casings were prepared for casting the concrete blocks and a mould release agent (Shell, 2004) was again smeared on the inside of the casings prior to pouring of the concrete.

The steel casings were prepared and positioned, along with twelve cylindrical casing specimens that were used for material property testing.

Preparation and location of the steel casings and cylindrical moulds prior to pouring of the concrete can be seen in Figure 95.
The concrete that was selected to represent the surrounding rockmass was reduced in strength from the previous Stage 1 tests. The Stage 1 test consisted of high strength 60 MPa concrete with 20 mm size aggregate and a nominal slump of 120 mm. For the Stage 2 tests, a 40 MPa concrete was used with 10 mm size aggregate and a nominal slump of 80 mm. 0.8 m³ of concrete was ordered to fill the steel casings and cylindrical moulds.

On arrival of the concrete onto site, it was noted that the concrete was not of high quality. Upon pouring the concrete into a wheelbarrow, the concrete was blocky, dry, warm and not free flowing. Due to our time constraints, we transferred the poured concrete back into the agitator and saturated the concrete mix. This was not a recommended practice.

The concrete was poured into the steel casings and cylindrical moulds as shown in Figure 96. A vibrator was used to remove air pockets in the concrete castings.
The concrete was left to cure with wet hessian sacks draped over the top of the specimens to control the curing process and avoid inconsistent reactions throughout the concrete as shown in Figure 97.
6.2. **Drilling and installation of reinforcing element**

Drilling and installation of the reinforcing elements were undertaken at Hydramatic Engineering in Newcastle.

The ARO rapid face roofbolter was used to drill and install the reinforcing element as shown in Figure 98.

![Figure 98 – Positioning of the rapid face roofbolter](image)

The boreholes of the first three samples S2-1, S2-2 and S2-3 were drilled with a 28 mm bit and to a depth of 1.125 m. These first three samples were installed with no pre-tension. One capsule of Minova slow-set resin at 600 mm in length was used for encapsulation.

Images of the Stage 2 drilling procedures are shown in Figures 99-101.
Figure 99 – Inserting drill steel prior to drilling sample

Figure 100 – Drilling borehole of Sample S2-2

Figure 101 – Close up of drilling the borehole of Sample S2-2
A summary of the drilling and installation of the first three rockbolts, including issues encountered are listed in Table 21.

Table 21 – Summary of the installation of samples S2-1, S2-2 & S2-3

<table>
<thead>
<tr>
<th>Sample #</th>
<th>Drilling</th>
<th>Installation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-1</td>
<td>Hole length: 1125 mm</td>
<td>Spin time: 20 s</td>
<td>Installed ok, did not hold rockbolt in long enough for resin to set</td>
</tr>
<tr>
<td>S2-2</td>
<td>Hole length: 1125 mm</td>
<td>Spin time: 20 s</td>
<td>Installed ok, did not hold rockbolt in long enough for resin to set. Had to thrust rockbolt in again</td>
</tr>
<tr>
<td>S2-3</td>
<td>Hole length: 1125 mm</td>
<td>Spin time: 20 s</td>
<td>Installed ok, resin nut breaking out quickly and spinning nut at end of thread</td>
</tr>
</tbody>
</table>

The installation and spinning of the rockbolts can be seen in Figures 102 and 103.

Figure 102 – Installation of rockbolt start
Samples S2-4, S2-5 and S2-6 were to have a pre-tension of 40 kN applied to each rockbolt but these were not attained. A mixture of fast-set and slow-set resin was required to induce a pre-tension load within the rockbolt. The resin included a 150 mm fast-set resin capsule, followed by a 400 mm slow-set capsule. It was planned that an Ultrasonic Transducer was to be used to measure the pre-tension applied to the rockbolt. The transducer transmits a sonic pulse at one end of the rockbolt and measures the time for the echo to reflect back from the opposite end of the rockbolt. The Ultrasonic Transducer was to be used in conjunction with a hydraulic load cell. Unfortunately the Ultrasonic Transducer was not supplied in time and a hydraulic load cell was only used during the tests.

The hydraulic load cell was placed between two steel plates that were located between the concrete surface (borehole collar) and the dome plate as shown in Figure 104.
There were issues encountered with regards to the installation of samples S2-4, S2-5 and S2-6. A summary of the installation problems encountered are listed in Table 22.

Table 22 – Summary of the installation of samples S2-4, S2-5 & S2-6

<table>
<thead>
<tr>
<th>Sample #</th>
<th>Drilling</th>
<th>Installation</th>
<th>Comments</th>
</tr>
</thead>
</table>
| S2-4     | Hole length: 1125 mm| Spin time: 20 s
Hold: 60 s
Pre-tension required: 40 kN
Pre-tension attained: 0 kN | Borehole was too long after the steel plates and load cells were introduced to the system. The length of the hole was too long to allow the rockbolt to secure itself to the fast set resin capsule. |
| S2-5     | Hole length: 1060 mm| Spin time: 20 s
Hold: 60 s
Pre-tension required: 40 kN | Two fast-set resin capsules were inserted into the borehole in order to gain an applied pre-tension. The resin nut broke out quickly once |
Pre-tension attained: 55 kN

the rockbolt made contact with the end of the borehole but with the presence of two fast-set resin capsules, still had sufficient anchoring to create a pre-tension

| S2-6 (Not Tested) | Hole length: 1065 mm | Spin time: 20 s  
|                  |                   | Hold: 90 s |
|                  |                   | Pre-tension required: 40 kN |
|                  |                   | Pre-tension attained: 0 kN |
|                  |                   | During installation, the fast-set resin did not harden sufficiently to secure the free end of the rockbolt. The rockbolt continued to rotate after sufficient time for the fast-set to cure. |
|                  |                   | The resin nut broke out quickly once the rockbolt made contact with the end of the borehole and led to insufficient mixing of the resin and catalyst. |

Figure 105 shows the failed installation of Sample S2-4 as the rockbolt could be pulled free from the resin encapsulation, along with the steel plates and hydraulic load cell.

![Figure 105 - Failure to apply pre-tension to the rockbolt, present with hydraulic load cell](image)

A closer examination of the rockbolts that failed to induce a pre-tension load can be seen in Figure 106. It would appear that there was sufficient resin
present for encapsulation, due to excess resin present at the borehole collar. It was probably therefore that the lack of pre-tension was due to either insufficient or over-mixing of the short, fast-set resin capsule.

![Image of rockbolt failing to induce pre-tension due to insufficient resin mixing](image)

**Figure 106 – Image of rockbolt failing to induce pre-tension due to insufficient resin mixing**

### 6.3. Instrumentation

The data acquisition system (DAQ) was modified for the Stage 2 tests. Two Solartron Mach 1 B25 transducers with Boxed Inline Conditioning Module (BICM) were used to measure shear displacement of the smaller specimen. These LVDTs were linked to the DAQ system as shown in Figure 107. Having a mobile DAQ system reduced the length of signal cables and hence limited the noise/stray currents in the signal cables.
The mobile data acquisition system (DAQ) included a line conditioner to reduce any noise in the 240V power supply and a 15 volt regulated power supply to power the LVDTs.

The LVDTs were mounted onto the top reaction beams using a magnetic base stand. The shear displacement was measured between the flanges of the steel casing containing the rock specimen as shown in Figure 108.
6.4. Testing procedure

All data from the Stage 2 tests were recorded by the DAQ. All samples were raised onto the test bench and manoeuvred into position with the use of an overhead chain block and guide beam system. It was essential that the test specimen was symmetric when placed in the shear test facility prior to applying any load.

Once a specimen was in place, a load was applied to raise the lower piston from the base of the cylinder housing. The pressure transducer was then zeroed. All channels in the DAQ were tested to ensure all instrumentation components were working and recording accurately as shown in Figure 109.

![Figure 109 – Testing DAQ channels and instrumentation prior to testing](image)

During each test, a relatively constant loading rate was applied. Each specimen had the loading rate calculated after the testing phase in both shear displacement (mm) per minute and load (kN) per minute.
During the Stage 1 and Stage 2 tests it was noted that the maximum shear displacement capacity of 50 mm in the shear testing facility was not sufficient to induce failure of the reinforcing element. At certain stages during a test, the shear load was removed from this specimen to allow packing to be introduced under the small specimen. This increased the maximum allowable shear displacement by 20 mm. The packing was introduced as early in the loading cycle as possible to avoid removing the applied load near or during the yield point of the reinforcing element.

6.5. Stage 2 test results

6.5.1. Reinforced Specimens

The Stage 2 samples were to have three samples with zero pre-tension on the reinforcing element and three with 40 kN tonnes applied to the reinforcing element. Due to issues in the installation procedure, the Stage 2 tests comprised four reinforced samples with zero pre-tension and one reinforced sample with 55 kN of pre-tension. Each reinforced sample was summarised as follows:

- S2-1 – Sample 1 of Stage 2 tests, zero pre-tension
- S2-2 – Sample 2 of Stage 2 tests, zero pre-tension
- S2-3 – Sample 3 of Stage 2 tests, zero pre-tension
- S2-4 – Sample 4 of Stage 2 tests, zero pre-tension
- S2-5 – Sample 5 of Stage 2 tests, 55 kN pre-tension

6.5.1.1. Sample S2-1

Sample S2-1 was the first of the Stage 2 tests that was undertaken 28 days after pouring the concrete. This sample was also the first test undertaken with the new instrumentation including LVDTs and the mobile DAQ system.
The sample was initially loaded to 170 kN with a corresponding displacement of 15 mm in order to insert a packing plate under the smaller specimen. The bolt relaxed however reducing the size of the gap. The load was re-applied to 227 kN which provided sufficient clearance to insert the 20 mm packing plate.

The shearing of the reinforced concrete samples during the test can be seen in Figures 110 and 111.

Figure 110 – An indication of the amount of shear displacement during testing – the two holes were initially aligned prior to commencement of the test.
A graph of the load-displacement curve was constructed based on the three loading cycles, as shown in Figure 112. The relaxation displacement was not recorded as the data acquisition was paused during this period. It was assumed that each loading cycle continued through the final loading point prior to the removal of the load from the system.

The load-displacement curve shown in Figure 112 seems to indicate two types of behaviour, elastic and inelastic, designated as zone 1 and zone 2 respectively. The maximum load in zone 1 was approximately 112 kN and the average stiffness in this zone was 22 kN/mm. In zone 2, the peak load was 313 kN and the average stiffness was 2 kN/mm. Results can be seen in Table 23. It should be noted that the test was terminated due to the limits of shear displacement rather than to shear failure of the rockbolt.
Figure 112 – Load-displacement combination graph for Sample S2-1

Table 23 – Results summary for Sample S2-1

<table>
<thead>
<tr>
<th>Loading Cycle</th>
<th>1-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-tension Applied</td>
<td>kN</td>
</tr>
<tr>
<td>Initial Shear Displacement</td>
<td>mm</td>
</tr>
<tr>
<td>Zone 1 Load</td>
<td>kN</td>
</tr>
<tr>
<td>Zone 1 Shear Displacement</td>
<td>mm</td>
</tr>
<tr>
<td>Zone 1 Stiffness</td>
<td>kN/mm</td>
</tr>
<tr>
<td>Peak Load</td>
<td>kN</td>
</tr>
<tr>
<td>Peak Shear Displacement</td>
<td>mm</td>
</tr>
<tr>
<td>Zone 2 Stiffness</td>
<td>kN/mm</td>
</tr>
<tr>
<td>Zero-Load Shear Displacement</td>
<td>mm</td>
</tr>
<tr>
<td>Loading Rate 1</td>
<td>kN/min</td>
</tr>
<tr>
<td>Loading Rate 2</td>
<td>mm/min</td>
</tr>
</tbody>
</table>

Minor alterations were made to the shear test facility, which increased the shear displacement capacity by to the removal of two steel plates that were used to increase the height of the bench. This alteration was only temporary and cannot be done during the initial loading process to gain the extra displacement. This
was repeated twice and in both cases a similar load-displacement curve was found as shown in Figure 113.

![Graph](image)

**Figure 113 – Load-displacement of S2-1 after reapplication of load after >55mm of shear displacement**

Figure 113 shows a symmetric path traced during loading and unloading of the test sample during one of the loading cycles.

After 75mm of shear displacement, Sample S2-1 had not failed in shear and the test was terminated.

An examination of the rockbolt indicated that it experienced a combination of both shear and axial forces that led to the creation of two plastic hinges at the points of maximum bending stress. With the lower strength concrete, the rockbolt was able to crush the concrete around the borehole wall relatively easier than in the Stage 1 tests as shown in Figure 114. The rockbolt was well bonded to the surrounding concrete through the resin encapsulation but did not allow the ultimate tensile strength of the rockbolt to be attained as shown in Figure 115.
This first test of the Stage 2 tests clearly showed the increase in shear displacement that the reinforcing element can sustain when installed into a softer rockmass with a relative lower UCS and Elastic Modulus. The extent of crushing and deformation of Sample S2-1 was dissimilar to the results in the Stage 1 tests that had an otherwise identical reinforcing element but encased in a relatively stronger rockmass.
6.5.1.2. Sample S2-2

Sample S2-2 was installed with no pre-tension applied to the rockbolt and testing was undertaken after 34 days of the concrete curing. During the initial loading of the test sample, there were large spikes in the recording data from the pressure transducer indicating an induced outside “noise”. It was concluded that the induced noise was due to the close proximity of the DAQ signal wires to the 240 V power cables. The two sets of cables were separated to prevent this recurrence.

A slight gap or opening appeared between the two shearing surfaces after a load >100 kN was applied to Sample S2-2 as shown in Figure 116.

![Figure 116 – Gap appearing along shear plane during shear test on S2-2](image)

The gap may have been due to the specimen not being positioned correctly prior to testing. The previous tested samples did not show any significant gap along the shear plane when subjected to load. The piston was assumed to have applied pressure in a non-symmetric plane due to the initial placement of the
sample. Once the bearing plates were introduced during the second loading phase, the gap between the shear planes reduced.

An illustration of the assumed mechanisms and reasons for the increased gap between the shear planes can be seen in Figure 117.
a. Initial non-symmetric placement of Sample S2-2

b. Gap appearing between the shear plane of Sample S2-2 after first phase loading

c. Realignment of smaller specimen once steel packing was introduced in the second loading phase of Sample S2-2

Figure 117 – Illustration of the issues associated with the two phase loading of Sample S2-2
During the two phase loading cycles, data was continually recorded including the unloading in the first loading cycle to determine the extent of the relaxation of the reinforced sample. Once 39.4 mm of shear displacement was attained the load from the system was withdrawn and the sample relaxed 10.5 mm to 28.9 mm. This indicated either some permanent deformation of the reinforcing element or failure of the rock-resin-rockbolt interface.

A steel plate was inserted and the LVDTs readjusted to increase the shear displacement capacity. The test continued with the second loading cycle. Combining the two loading cycles, the load-displacement curves (for both the left and right LVDTs) for Sample S2-2 is shown in Figure 118.

A total displacement of 53 mm was measured before the test was stopped. This corresponds to a maximum load of 240 kN. A summary of the results can be seen in Table 24.
Table 24 – Results summary for Sample S2-2

<table>
<thead>
<tr>
<th>Loading Cycle</th>
<th>1-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-tension Applied</td>
<td>kN</td>
</tr>
<tr>
<td>Initial Shear Displacement</td>
<td>mm</td>
</tr>
<tr>
<td>Zone 1 Load</td>
<td>kN</td>
</tr>
<tr>
<td>Zone 1 Shear Displacement</td>
<td>mm</td>
</tr>
<tr>
<td>Zone 1 Stiffness</td>
<td>kN/mm</td>
</tr>
<tr>
<td>Peak Load</td>
<td>kN</td>
</tr>
<tr>
<td>Peak Shear Displacement</td>
<td>mm</td>
</tr>
<tr>
<td>Zone 2 Stiffness</td>
<td>kN/mm</td>
</tr>
<tr>
<td>Zero-Load Shear Displacement</td>
<td>mm</td>
</tr>
<tr>
<td>Loading Rate 1</td>
<td>kN/min</td>
</tr>
<tr>
<td>Loading Rate 2</td>
<td>mm/min</td>
</tr>
</tbody>
</table>

The first loading cycle increases at a relatively faster rate between 5 to 10 mm of shear displacement and then the slope of the curve reduces. Prior to 14 mm of shear displacement, there was a reduction in load applied to the sample. It was likely that this indicated slipping of the rockbolt between the resin-rockmass or rockbolt-resin interface whereby the applied load reduced from 115 kN to 103 kN.

It was interesting to note that beyond this load the system displayed a constant positive stiffness of approximately 3.7 kN/mm. This was unlike what has been observed in axial loading of a rockbolt where axial load reduces with displacement (Hagan & Weckert, 2004).

On reapplication of load in the second loading cycle, the load quickly increased with displacement until the final load in the first cycle. Beyond this the load-displacement curve continued with the same characteristic as in the first cycle.
Figure 119 shows the loading rate of Sample S2-2 in terms of load (kN) per minute and shear displacement (mm) per minute. Figure 119 shows that the rate for both curves becomes constant beyond 14 mm.

![Graph showing loading rates for Sample S2-2 on first loading cycle](image)

Figure 119 – Loading rates for Sample S2-2 on first loading cycle

While Sample S2-2 was deformed there was no observable failure or appreciable necking in the rockbolt as shown in Figure 120.

![Image of Sample S2-2 rockbolt subject to shear loading](image)

Figure 120 – Side profile of Sample S2-2 rockbolt subject to shear loading

The crushing around the borehole was relatively minimal as seen in Figure 121.
6.5.1.3. **Sample S2-3**

Sample S2-3 was loaded at a rate of 72 kN per minute, which was a higher rate than the previous samples.

Values of applied load (pressure transducer) and shear displacement (LVDT) were recorded at a rate of 100 samples per second (median of 10 samples at a sampling rate of 1000 samples per second).

During each of the loading phases of the sample, there was little perceptible opening between the two sample blocks. Extra care was taken when positioning the sample to ensure the sample was flush against the positioning jig at the end of the lower steel block. Figure 122 shows the near-parallel shearing surfaces of the small and large specimen during the final loading cycle.
The test involved four loading cycles to a maximum shear displacement of 87 mm.

Similar to samples S2-1 and S2-2, the load-displacement curve exhibited two zones, as shown in Figure 123. The maximum load sustained in Zone 1 was approximately 140 kN and the stiffness was 12 kN/mm. In Zone 2, the stiffness reduced to 2 kN/mm. The load peaked at 226 kN at the end of the first loading cycle.

On reapplication of the load in cycle 2, the initial stiffness was 21 kN/mm, much greater than that observed in the first loading cycle.

Unlike Sample 2-2, a much higher load was sustained which peaked at 338 kN after a shear displacement of 66mm. This was of similar order of magnitude to the levels observed in S2-1. The peak load in cycles 3 and 4 increased only slightly to 361 kN and 362 kN respectively. A summary of the results can be seen in Table 25.
The difference in observed behaviour to Sample S2-2 may be accounted to that the first cycle was displaced over 45 mm and this initiated crushing around the borehole which was common in the previous tests. Since the second phase was loaded at a high loading rate, the ‘slack’ was taken up very quickly and continued well past the final applied load in the first loading phase.

Figure 123 – Load-displacement graph of the loading cycles for Sample S2-3

Table 25 – Results summary for Sample S2-3

<table>
<thead>
<tr>
<th>Loading Cycle</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-tension Applied</td>
<td>kN</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial Shear Displacement</td>
<td>mm</td>
<td>0.0</td>
<td>40.2</td>
<td>58.4</td>
</tr>
<tr>
<td>Zone 1 Load</td>
<td>kN</td>
<td>139</td>
<td>276</td>
<td>340</td>
</tr>
<tr>
<td>Zone 1 Shear Displacement</td>
<td>mm</td>
<td>10.9</td>
<td>54.8</td>
<td>70.9</td>
</tr>
<tr>
<td>Zone 1 Stiffness</td>
<td>kN/mm</td>
<td>12.4</td>
<td>21.1</td>
<td>39.4</td>
</tr>
<tr>
<td>Peak Load</td>
<td>kN</td>
<td>226</td>
<td>338</td>
<td>361</td>
</tr>
<tr>
<td>Peak Shear Displacement</td>
<td>mm</td>
<td>45.0</td>
<td>65.7</td>
<td>76.9</td>
</tr>
<tr>
<td>Zone 2 Stiffness</td>
<td>kN/mm</td>
<td>2.2</td>
<td>5.6</td>
<td>3.6</td>
</tr>
<tr>
<td>Zero-Load Shear Displacement</td>
<td>mm</td>
<td>40.2</td>
<td>58.4</td>
<td>67.3</td>
</tr>
<tr>
<td>Loading Rate 1 (range)</td>
<td>kN/min</td>
<td>53-66</td>
<td>59-71</td>
<td>82-119</td>
</tr>
<tr>
<td>Loading Rate 2 (range)</td>
<td>mm/min</td>
<td>3.0-10.7</td>
<td>3.1-4.9</td>
<td>2.3-2.9</td>
</tr>
</tbody>
</table>
Several cracks developed in the smaller specimen and extended parallel to the rockbolt. Figure 124 shows the formation of the cracks in the smaller specimen after removal of the steel casing.

Figure 124 – Formation of cracks within the S2-3 smaller specimen sample

The side profile of the rockbolt can be seen in Figure 125, with the extent of crushing around the borehole in Figure 126.

Figure 125 – Side profile of Sample S2-3
Figure 126 – Extent of crushing around the borehole of Sample S2-3

A sketch of the crushed zone and deformation of the rockbolt can be seen in Figure 127.

a) Measurements around the borehole crushing extremities
b) Measurements of the deformation of the rockbolt

Figure 127 – Measurements of the extent of breakout around the rockbolt of Sample S2-3

Plastic hinges were developed within the reinforcing element and there was evidence of yielding and necking. This interaction of the element within the concrete rockmass and resin encapsulation can be seen in Figure 128.

Figure 128 – Extent of shearing within Sample S2-3
6.5.1.4. **Sample S2-4**

Sample S2-4 was the first of the reinforcing elements that was to have a pre-tension applied. Due to issues during installation however, the sample was installed with no pre-tension applied. It should also be noted that the rockbolt did not have a nut and bearing plate.

Testing on Sample S2-4 was undertaken at a low loading rate. The sample was loaded in two stages with the duration of each loading cycle being 140 minutes and 87 minutes. The initial loading phase of the first cycle had an average stiffness of 16.8 kN/mm up to 130 kN. In the second loading phase, the average initial loading stiffness was 17.1 kN/mm.

In the first loading cycle, beyond the initial elastic range of the curve, the graph displayed a consistent, near-flat, load-displacement relationship between 20 mm and 45mm of shear displacement at a load of 170 kN as shown in Figure 129.

Later inspection of the rockbolt indicated that there was failure at the rockbolt-resin-rockmass interface. The constant load represents the sum of the friction to drag the rockbolt through the rock and the force to deform the rockbolt
In the second loading cycle, the peak load was only 141 kN, much less than that achieved in the first cycle.

The loading rate applied to Sample S2-4 is shown in Figure 130. The loading rate was increased once it became obvious there was failure at the rockbolt-resin-rockmass interface but it was still at a lower rate compared to the previous samples. A summary of the results can be seen in Table 26.
Table 26 – Results summary for Sample S2-4

<table>
<thead>
<tr>
<th>Loading Cycle</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-tension Applied</td>
<td>kN</td>
<td>0.0</td>
</tr>
<tr>
<td>Initial Shear Displacement</td>
<td>mm</td>
<td>0.0</td>
</tr>
<tr>
<td>Zone 1 Load</td>
<td>kN</td>
<td>131</td>
</tr>
<tr>
<td>Zone 1 Shear Displacement</td>
<td>mm</td>
<td>8.3</td>
</tr>
<tr>
<td>Zone 1 Stiffness</td>
<td>kN/mm</td>
<td>16.6</td>
</tr>
<tr>
<td>Peak Load</td>
<td>kN</td>
<td>172</td>
</tr>
<tr>
<td>Peak Shear Displacement</td>
<td>mm</td>
<td>43.0</td>
</tr>
<tr>
<td>Zone 2 Stiffness</td>
<td>kN/mm</td>
<td>0.1</td>
</tr>
<tr>
<td>Zero-Load Shear Displacement</td>
<td>mm</td>
<td>32.3</td>
</tr>
<tr>
<td>Loading Rate 1 (range)</td>
<td>kN/min</td>
<td>1.7-2.3</td>
</tr>
<tr>
<td>Loading Rate 2 (range)</td>
<td>mm/min</td>
<td>0.1-0.5</td>
</tr>
</tbody>
</table>

In an examination of the reinforcing element, it was found that the fast-set resin cartridge did not rupture sufficiently to allow a pre-tension to be generated in the element as seen in Figure 131. The fast-set resin cartridge was also not left long enough to allow the catalyst to mix and harden the resin.

![Figure 131 – Side profile of Sample S2-4 showing the encapsulation along the length of the reinforcing element](image)

The reinforcing element displayed the formation of the plastic hinges and localised crushing around the borehole as seen in Figure 132.
There was greater deformation of the reinforcing element in the fast loading Sample S2-3 compared to the slow-loading of Sample S2-4.

6.5.1.5. Sample S2-5

Sample S2-5 was the only reinforced specimen in this series that had a pre-tension applied to the reinforcing element. During installation, two fast-set resin capsules were inserted into the borehole (as opposed to one in Sample S2-4) to ensure a pre-tension could be applied.

Excess resin was observed between the bearing plate and collar of the borehole. A 60 kN pre-tension was applied to the rockbolt that reduced to around 55 kN after removing the torque wrench. The resin was allowed to set for 30 minutes before the end nut and hydraulic load cell were removed from the end of the rockbolt.

Sample S2-5 was tested in four loading cycles with a total shear displacement of 86 mm as seen in Figure 133.

In the first loading cycle, the peak load and average stiffness were:

- Zone 1 – 150 kN and 23 kN/mm
- Zone 2 – 303 kN (at 42 mm shear displacement) and 3.6 kN/mm
In the second loading cycle, the peak load reached only 256 kN after 20 mm of shear displacement. The initial stiffness between 50 kN and 150 kN was inherently much higher than in the first load cycle of 38 kN/mm. Thereafter the average stiffness of 3 kN/mm was similar to that in cycle 1. The third and fourth loading cycles allowed a slight increase in shear displacement of 10 mm that reached peak loads of 182 kN before the test was terminated. A summary of the results can be seen in Table 27.

![Load-displacement graph for Sample S2-5 (four loading cycles)](image)

Table 27 – Results summary for Sample S2-5

<table>
<thead>
<tr>
<th>Loading Cycle</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-tension Applied</td>
<td>kN</td>
<td>55</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial Shear Displacement</td>
<td>mm</td>
<td>0.0</td>
<td>35.3</td>
<td>57.2</td>
</tr>
<tr>
<td>Zone 1 Load</td>
<td>kN</td>
<td>204</td>
<td>203</td>
<td>173</td>
</tr>
<tr>
<td>Zone 1 Shear Displacement</td>
<td>mm</td>
<td>11.5</td>
<td>45</td>
<td>67.8</td>
</tr>
<tr>
<td>Zone 1 Stiffness</td>
<td>kN/mm</td>
<td>23.4</td>
<td>37.8</td>
<td>25.1</td>
</tr>
<tr>
<td>Peak Load</td>
<td>kN</td>
<td>303</td>
<td>256</td>
<td>180</td>
</tr>
<tr>
<td>Peak Shear Displacement</td>
<td>mm</td>
<td>42.5</td>
<td>66.4</td>
<td>74.1</td>
</tr>
<tr>
<td>Zone 2 Stiffness</td>
<td>kN/mm</td>
<td>3.6</td>
<td>2.7</td>
<td>1.0</td>
</tr>
<tr>
<td>Zero-Load Shear Displacement</td>
<td>mm</td>
<td>35.3</td>
<td>57.2</td>
<td>63.4</td>
</tr>
<tr>
<td>Loading Rate 1 (range)</td>
<td>kN/min</td>
<td>71</td>
<td>64-77</td>
<td>53-77</td>
</tr>
<tr>
<td>Loading Rate 2 (range)</td>
<td>mm/min</td>
<td>2.8-9.8</td>
<td>2.6-6.9</td>
<td>3.9-6.0</td>
</tr>
</tbody>
</table>
The loading rate in each cycle was similar at between 60 and 80 kN/minute.

There was evidence that the extremity of the resin encapsulation had been subjected to some stress. The resin around the rockbolt had changed in colour to a white ring as seen in Figure 134. This indicated an increase in stress within the resin encapsulation as it resisted axial movement of the rockbolt.

![Figure 134 – Reinforced Sample S2-5 with excess resin smeared about the outer surface of the concrete test specimen](image)

The extent of breakout around the borehole in the larger specimen can be seen in Figure 135 and schematically indicating the extremity measurements in Figure 136.

![Figure 135 – Extent of crushing around the borehole prior to failure of Sample S2-5](image)
a) Measurements around the borehole crushing extremities

Figure 136 – Measurements of the extent of breakout around the rockbolt of Sample S2-5

b) Measurements of the deformation of the rockbolt

Sample S2-5 was well encapsulated along the entire length of the reinforcing element as shown in Figure 137 and maintained a stiff load-deformation loading rate until load reached 200 kN. There were no evidence of necking within the rockbolt even though the load applied exceeded 300 kN.
6.5.2. Summary

The Stage 2 tests were undertaken to understand the capabilities and limitations of the shear test facility after improvements were made to the test facility and measurement instrumentation.

Rockbolts were installed in five concrete samples by a rapid-face rockbolter with the intention of applying a pre-tension load in three of the samples. There were minor installation problems that resulted in only one sample having a pre-tension of 55 kN.

Each test involved two or more loading and unloading cycles due to the limited travel of the shear test facility. After unloading, spacer plates were added to increase the travel capacity. Both load and shear displacements were measured in the loading and unloading stages of each cycle.

The results of the Stage 2 tests are summarised in Table 28 and a detailed summary can be found in appendix A1.
Table 28 – Summary of Stage 2 tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Loading Cycles</th>
<th>Pre-tension applied</th>
<th>Peak shear load</th>
<th>Peak shear displacement</th>
<th>Reinforcing element failed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kN</td>
<td>kN</td>
<td>mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S2-1</td>
<td>4</td>
<td>0</td>
<td>313</td>
<td>54.7</td>
<td>No</td>
</tr>
<tr>
<td>S2-2</td>
<td>4</td>
<td>0</td>
<td>240</td>
<td>53.1</td>
<td>No</td>
</tr>
<tr>
<td>S2-3</td>
<td>4</td>
<td>0</td>
<td>362</td>
<td>86.9</td>
<td>No</td>
</tr>
<tr>
<td>S2-4</td>
<td>2</td>
<td>0</td>
<td>172</td>
<td>63.8</td>
<td>No</td>
</tr>
<tr>
<td>S2-5</td>
<td>4</td>
<td>55</td>
<td>303</td>
<td>84.7</td>
<td>No</td>
</tr>
</tbody>
</table>

Over the five tests that were undertaken, the maximum peak load reached was 362 kN with Sample S2-3. This peak load was attained over shearing the two blocks 87 mm at a relatively high loading rate that reached up to 125 kN/min. In contrast Sample S2-4 reached a peak load of 172 kN over a shear displacement of 64 mm. Sample S2-4 was loaded at relatively low rates that averaged approximately 2.0 kN/min. With a pre-tension of 55 kN applied to Sample S2-5, a peak load of 303 kN was attained after the initial 43 mm of shear displacement. With this pre-tension applied, the load at the zone 1 transition point was 204 kN compared to a range of 94 to 139 kN for the previous Stage 2 tests. The stiffness up to the zone 1 transition point during the first loading cycles ranged from 14.1 to 23.4 kN/mm throughout the tests, with the highest stiffness reading produced by Sample S2-5.

None of the rockbolts failed in shear in this test series and very few showed visual signs of yielding. A lower strength concrete was used in this series with an average UCS and Young’s Modulus of 46 MPa and 33 GPa respectively.
6.6. Conclusions and Recommendations

A major element evident from the Stage 2 tests was the influence of the strength of the concrete (UCS and Young’s Modulus) on the shear performance of the rockbolt. The rockbolt sheared in the Stage 1 tests at a shear displacement of approximately 45 mm. Due to the inability to measure shear displacement during a test in Stage 1 it was not possible to monitor the load-displacement characteristic. In the Stage 2 tests, displacements of up to 87 mm were recorded though the rockbolt did not fail in shear. There was more crushing around the borehole wall observed compared to the Stage 1. No face plate and nut was installed to the rockbolt.

A peak load of 362 kN was attained in some tests. The maximum load in the elastic range in nearly all the tests was 150 kN.

Each test involved two or more loading and unloading cycles. Each load-displacement curve exhibited similar characteristics with a much stiffer initial loading phase in the second and subsequent loading cycles. There was a distinct transition point where the stiffness of the curve dramatically decreased and remained relatively constant. Each subsequent loading cycle showed almost identical loading characteristics with an initial stiff loading phase followed by a sudden change to a less stiff phase on the load-displacement curve.

When comparing two similar rockbolts under two different loading conditions, the nature of the load-displacement curve dramatically alters. With a relative faster loading rate, each subsequent loading cycle increased the peak loads that were attained until the load was removed from the system. Each subsequent loading cycle had a much higher stiffness over a shorter shear displacement. The much lower loading rate applied to a reinforced sample over two cycles displayed a reduction in the peak load attained after the first cycle. The sample maintained a consistent load after the initial elastic range of the curve was overcome in which a consistent, flat, load-displacement relationship was created over a majority of the allowable shear displacement.
Upon visual inspection of the reinforcing elements after subject to the shear load, an expected ‘s’ shape was created at two symmetric points where bending moment was at a maximum. There was only evidence of the element that was loaded at a fast loading rate (S2-3) with the commencement of yielding and necking of the element.

The first recommendation from the Stage 2 testing program was to alter the shear testing facility to increase the shear displacement capacity. This required alteration of the lower steel block and removal of two of the three steel plate sections of the bench.

The instrumentation intended for data acquisition was not capable of acquiring appropriate strain-gauge rockbolt data. New data acquisition hardware and software was installed.
7. Stage 3 Tests

The Stage 3 tests were undertaken to analyse the effects of pre-tension performance of the rockbolt, subjected to shear forces. A total of six rockbolts were installed and tested including three standard JBX bolts (as per Stage 2 tests) with different pre-tension levels and three strain-gauge rockbolts supplied by SCT Pty Ltd were also installed with different pre-tensions.

The strain-gauge rockbolts were used to determine the load distribution along the rockbolt. Each reinforcing element was installed in concrete with a higher UCS strength than that used in the Stage 2 tests but comparable to the Stage 1 tests.

Further modifications were made to the shear test facility, including removal of the inside vertical support beam to allow greater access to the rockbolt, face plate and nut at the collar of the hole as is shown in Figure 138. This also allowed access to the strain-gauged rockbolts.

Figure 138 – Improving access to the reinforced sample by removal of the central section of the vertical support beam
A new lower profile steel block was fabricated to increase the shear displacement capacity of the test rig to 65mm, an increase of 20mm. The steel block incorporated a step that supported on one side the smaller of the two concrete specimens as shown in Figure 140.

![Figure 139 – Introduction of the modified lower steel block](image)

7.1. **Concrete casting**

As in the Stage 2 tests, the concrete was poured into steel castings in the School of Civil and Environmental Engineering at the University of New South Wales as shown in Figure 140. Twelve cylindrical specimens were cast in conjunction with the test specimens to determine the concrete strength and material properties.
The concrete was found to have a strength 60 MPa. The concrete had 20 mm size aggregate and a nominal slump of 120 mm. 0.8 m$^3$ of concrete was used to fill all the steel casings and cylindrical moulds.

Each test sample was vibrated to remove any air pockets that may have formed during pouring. The cylindrical moulds were cast on a vibrating table for a consistent and suitable testing sample.

Wet hessian sacks were draped over the top of the specimens during curing to ensure consistent curing of the concrete as seen in Figure 141.
Three custom strain gauged rockbolts were manufactured by SCT Pty Ltd specifically for this project from the same batch of rockbolts that were used in the three series of tests.

Nine pairs of diametrically opposed strain gauges were mounted along the length of the rockbolt to determine changes in axial load and bending strain profiles on either side of the shear plane. The stain gauges were positioned as close to the shear plane as possible. Figure 142 shows the dimensions of the installed rockbolt in the rockmass and the locations of the strain gauges in relation to the shear plane. Due to limitation in the date recording system, only strain-gauge 1 to 8 as indicated in Figure 143 were mounted.
a) Schematic diagram of the dimensions of the reinforced sample and the position of the reinforcing element in the concrete

b) Schematic diagram of the location of the strain gauges in the rockbolt and the distances away from the shear plane

Figure 142 – Schematic diagrams showing the location of the strain gauges in the reinforcing element in relation to the shear plane
The strain gauges had a nominal resistance of 120Ω and the gauge factor of 2.065 ± 0.5% at 24˚C.

7.2.1. Calibration

Each strain-gauged rockbolt was calibrated prior to installation and testing of the reinforced sample. The strain-gauged rockbolts were loaded into a 500 kN Avery Universal Testing Machine (UTM).

The DAQ system was configured to record strain gauge readings at nominated loads applied to the rockbolt. Three readings in units of strain and three readings in output volts were recorded for each of the 16 strain-gauges along the rockbolt. Each rockbolt was loaded in 20 kN increments up to 100 kN. A maximum calibration load of 100 kN was selected as this load was well within the elastic range of the rockbolt.

A regression analysis was used to determine the relationship between the strain recordings and applied load throughout the rockbolt.

Figure 143 shows an example of the change in strain with applied load. Each step indicates the increase in strain with applied load. The graph shows a clear increase in strain with applied axial load. A linear regression analysis can be calculated between the applied axial load and strain; this is shown in Figure 143b.
There was significant variation (noise) in the 240V power supply within the laboratory that resulted in minor variability in the strain gauge readings. This noise was enhanced in the latter part of the day. Calibration of the strain gauges was undertaken in minimal time so this time-based interference did not have a major influence on the calibration of the strain gauges.
7.3. **Drilling and installation of reinforcing element**

Three standard JBX rockbolts and three strain gauge bolts were installed at Hydramatic Engineering. All three standard rockbolts were to be installed with a pre-tension applied to the rockbolt and as well as two of the strain-gauged rockbolts. The other strain-gauged rockbolt was to be installed with no pre-tension applied in order to compare the distribution of load encountered in the strain-gauged rockbolts with different levels of pre-tension.

The ARO rapid face roofbolter was set-up and positioned appropriately ready for drilling and installation of the reinforcing element. Test samples were positioned as shown in Figure 144 and assembled as shown in Figure 145 on the drilling bench using a crane and magnetic lifting block.

![Figure 144](image)

**Figure 144** – Positioning of the concrete samples using a magnetic lifting block.

The steel casings of the test samples were spot welded to the steel drilling bench.
Sample S3-1 was drilled with a 28 mm roof bit to a depth of 1.065m. Initially, automatic thrust control was used in drilling however the resulted in failure of the drill steel near the drill chuck as shown in Figure 146. Subsequent drilling was completed in manual mode.
It was intended that a pre-tension was to be applied to Sample S3-1 using a mixture of fast-set and slow-set resins. The resin capsules inserted into the borehole included a 150 mm fast-set resin capsule followed by a 400 mm slow-set resin capsule.

A hydraulic load cell was placed between the two steel plates located between the concrete surface (borehole collar) and the dome washer as seen in Figure 147.

![Figure 147 – Positioning of the load cell between two steel plates](image)

Due to issues associated with drilling, the borehole was reamed out to greater than the 28 mm and there was insufficient resin to encapsulate the far end of the rockbolt. This resulted in no pre-tension applied to the rockbolt. In future drilling of the borehole into the concrete specimens it was recommended to measure the diameter of the borehole.

Standard JBX rockbolts were used in Samples S3-2 and S3-3 and both were installed with a pre-tension applied. The rotational torque in the drillhead of the roofbolter was insufficient to apply the desired pre-tension of 100 kN. The pre-tension was applied manually. Two fast-set resin capsules were inserted into
the borehole to ensure point anchorage occurred before the pre-tension was applied. The nut, bearing plate and load cell were left secured to the rockbolt for 15 to 20 minutes to ensure the slow-set resin had set and locked in the pre-tension.

Images of the Stage 3 drilling procedures can be seen in Figures 148 and 149.

![Figure 148 – Drilling of Sample S3-1](image)

As with the Stage 2 tests, there were some minor installation issues which are summarised in Table 29.
Table 29 – Summary of samples S3-1, S3-2 & S3-3 installation observations

<table>
<thead>
<tr>
<th>Sample #</th>
<th>Drilling</th>
<th>Installation</th>
<th>Issues</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-1</td>
<td>Hole length: 1065mm Spin time: 20s Hold: 60s Pre-tension required: 50kN Pre-tension attained: 0kN</td>
<td>The diameter of the borehole was too large from issues with drilling to allow the rockbolt to secure itself to the fast set resin capsule. The drill steel failed and was replaced. Automatic thrust on the roofbolter was too great to allow sufficient penetration.</td>
<td></td>
</tr>
<tr>
<td>S3-2</td>
<td>Hole length: 1065mm Spin time: 20s Hold: 60s Pre-tension required: 50kN Pre-tension attained: 38kN</td>
<td>Two fast-set resin capsules were inserted into the borehole in order to gain an applied pre-tension. The roofbolter could not provide sufficient torque to gain the required pre-tension.</td>
<td></td>
</tr>
<tr>
<td>S3-3</td>
<td>Hole length: 1065mm Spin time: 20s Hold: 60s Pre-tension required: 100kN Pre-tension attained: 55kN</td>
<td>Two fast-set resin capsules were inserted into the borehole in order to gain an applied pre-tension. The roofbolter could not provide sufficient torque to attain the required pre-tension. The rockbolt was pre-tensioned by hand until the slow-set resin set.</td>
<td></td>
</tr>
</tbody>
</table>

The strain-gauged bolts were then installed after the first three standard rockbolts were installed as seen in Figure 150.
There were issues encountered in regards to installation of the strain-gauged rockbolts in samples S3-4, S3-5 and S3-6. Two fast-set resin capsules were inserted into each of the boreholes in order to ensure the pre-tension could be applied. In order to gain an understanding of the axial and bending strain profiles, it was recommended that the strain-gauges were aligned in a vertical plain when installed in the test sample. This allowed analysis of the strains occurring on opposite sides of the rockbolt with expected zones of tension and compression.

The strain-gauged rockbolts were designed with a shear pin installed through the steel bar and the end nut. The installation process for this system commences with the rockbolt initially spun and consequently ruptures the resin capsules to mix the resin and catalyst. Once the resin was sufficiently mixed, the dolly must be withdrawn to allow removal of the shear pin and to allow the nut to be tightened and apply a pre-tension if required. After the first attempt in
The installation of the strain-gauged rockbolt, lines were marked in the dolly chuck as to where the strain gauges were located.

A summary of the installation results and issues encountered can be seen in Table 30.

**Table 30 – Summary of strain-gauged samples S3-4, S3-5 & S3-6 installation observations**

<table>
<thead>
<tr>
<th>Sample #</th>
<th>Drilling</th>
<th>Installation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-4 (ID 345)</td>
<td>Hole length: 1160mm</td>
<td>Spin time: 20s Hold: 60s Pre-tension required: 100kN Pre-tension attained: 0kN</td>
<td>The dome-ball and washer were not correctly utilised and the resin had set prior to any pre-tension. The roofbolter could not provide sufficient torque to gain the required pre-tension. The strain-gauges in the rockbolt were orientated horizontally and not in the recommended vertical direction.</td>
</tr>
<tr>
<td>S3-5 (ID 347)</td>
<td>Hole length: 1160mm</td>
<td>Spin time: 20s Hold: 60s Pre-tension required: 100kN Pre-tension attained: 54kN</td>
<td>During installation, the dolly chuck sheared not allowing sufficient pre-tension to be applied by the roofbolter. Fortunately the dolly chuck sheared off with the strain gauges position in the required vertical direction. The rockbolt had further pre-tension applied manually but there was insufficient time to gain the required pre-tension before the slow-set resin set.</td>
</tr>
<tr>
<td>S3-6 (ID 346)</td>
<td>Hole length: 1160mm</td>
<td>Spin time: 20s Hold: 60s Pre-tension</td>
<td>The rockbolt was installed slightly off-centre and once again the roofbolter could not provide sufficient torque to gain the required pre-tension.</td>
</tr>
</tbody>
</table>
Drilling and installation of the reinforced samples had varied results but ultimately achieved the desired objective. The major issues were in applying a pre-tension on the rockbolts within the sufficient time frame prior to the slow-set resin setting. The roof bolter did not have sufficient torque to achieve the desired pre-tension of 100 kN and there was insufficient time in achieving this level of pre-tension manually.

Inexperienced roofbolter operators and the failure of two drill steels as well as a dolly chuck exemplify some of the issues that were encountered during the installation phase.

### 7.4. Instrumentation

The data acquisition system (DAQ) was modified to accommodate the use of strain-gauged rockbolts. An additional interface card was installed to allow for the required additional channels and configured to include 16 strain-gauge channels and four channels for the pressure transducers and LVDTs. The new configuration of the shear testing facility is shown in Figure 151.
During the Stage 2 tests it was concluded that a plate and hydraulic load cell should be placed at the collar of the hole during the testing process. A pressure transducer was incorporated into the load cell and connected to the DAQ as shown in Figure 152.
The load cell and pressure transducer were calibrated using a 500 kN Avery Universal Testing Machine (UTM), applying a load at set increments. An initial reading was taken with no load applied and then progressed with 20 kN incremental loads up to a maximum load of 200 kN.

A linear regression analysis was undertaken on the voltage output of the pressure transducer and applied load of the UTM.

### 7.5. Testing procedure

The Stage 3 testing program commenced with testing of the standard JBX rockbolts S3-1, S3-2 and S3-3, followed by testing of the strain-gauged rockbolts S3-4, S3-5 and S3-6.

Sample S3-1 was used to test the additional instrumentation and modifications to the shear testing facility.

The modified shear testing facility had a single pass shear displacement capacity of 65 mm with the potential for shear displacements up to 105 mm with the use of additional packing plates.

A load cell was placed at the collar of the test samples, identical to the arrangement during the installation process, to measure the load between the bearing plate and concrete surface.
7.6. **Stage 3 test results**

7.6.1. Reinforced Specimens

All six reinforced test samples were intended to have some pre-tension applied up to a maximum of 100 kN. Each reinforcing element was installed in a relatively strong concrete with a UCS of 70 MPa. The six test samples can be summarised as follows:

- S3-1 – Sample 1 of Stage 3 tests: JBX rockbolt, zero pre-tension
- S3-2 – Sample 2 of Stage 3 tests: JBX rockbolt, 30 kN pre-tension
- S3-3 – Sample 3 of Stage 3 tests: JBX rockbolt, 70 kN pre-tension
- S3-4 – Sample 4 of Stage 3 tests: strain-gauged JBX rockbolt (serial number 345), zero pre-tension
- S3-5 – Sample 5 of Stage 3 tests: strain-gauged JBX rockbolt (serial number 347), 54 kN pre-tension
- S3-6 – Sample 6 of Stage 3 tests: strain-gauged JBX rockbolt (serial number 346), 40 kN pre-tension

7.6.1.1. **Sample S3-1**

Sample S3-1 was the first of the Stage 3 tests undertaken with the reinforcing element incorrectly installed due to an increased diameter borehole drilled. The element was not fully encapsulated in the borehole and therefore used to test the performance of the shear test facility after modifications were made to the facility and instrumentation.

Sample S3-1 is shown positioned and ready for testing in Figure 153.
The reinforced sample was tested as per the Stage 2 tests. After reaching a peak load of 166 kN and 49.5 mm of shear displacement the rockbolt had not sheared but the anchorage had clearly failed as shown in Figure 154. There was no end plate or nut secured on the rockbolt at the collar end of the borehole to resist the rockbolt to be pulled within the resin encapsulation.

Like in Stage 2, even though encapsulated had failed the rockbolt was still able to resist shear forces. A summary of the test results can be seen in Table 31.
Table 31 - Results summary for Sample S3-1

<table>
<thead>
<tr>
<th>Loading Cycle</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-tension Applied</td>
<td>kN 0.0</td>
</tr>
<tr>
<td>Initial Shear Displacement</td>
<td>mm 0.0</td>
</tr>
<tr>
<td>Zone 1 Load</td>
<td>kN 110</td>
</tr>
<tr>
<td>Zone 1 Shear Displacement</td>
<td>mm 8.0</td>
</tr>
<tr>
<td>Zone 1 Stiffness</td>
<td>kN/mm 18.1</td>
</tr>
<tr>
<td>Peak Load</td>
<td>kN 166</td>
</tr>
<tr>
<td>Peak Shear Displacement</td>
<td>mm 49.5</td>
</tr>
<tr>
<td>Zone 2 Stiffness</td>
<td>kN/mm 1.2</td>
</tr>
<tr>
<td>Zero-Load Shear Displacement</td>
<td>mm -</td>
</tr>
<tr>
<td>Loading Rate 1 (range)</td>
<td>kN/min 4.2-11.7</td>
</tr>
<tr>
<td>Loading Rate 2 (range)</td>
<td>mm/min 2.0-9.5</td>
</tr>
</tbody>
</table>

The system stiffness was 18 kN/mm up until the shear load reached 110 kN. Beyond this, a lower though constant stiffness of 1.2 kN/mm until the peak load of 166 kN was reached. During the loading of the reinforced sample there were minor ‘bumps’ in the graph due to the rockbolt slipping within the encapsulation and re-securing itself. The poor encapsulation and therefore bonding characteristics between the rockbolt, resin and surrounding rockmass enabled the rockbolt to pulled-out at the free-end of the sample and minimise the transfer of shear load through the rockbolt itself.

Since there was no end plate or nut secured on the rockbolt, the smaller specimen was seen to rotate away from the larger specimen as seen in Figure 155. The rotating mechanism was enhanced due the deficient installation of the rockbolt, which led to a significantly larger borehole and insufficient resin encapsulation.
7.6.1.2. **Sample S3-2**

Sample S3-2 was positioned and tested with a nut and plate combination secured at the free-end of the rockbolt. A load cell was inserted between the two additional steel plates similar to the orientation used when the rockbolts were initially installed into the concrete rockmass as shown in Figure 156. The load cell allowed manual recording of the applied load at the collar of the borehole and analysis of the load at the collar with respect to the applied shear load upon the reinforced sample.
The load cell was manually recorded at select intervals and plotted with the corresponding applied load reading that was attained from the data acquisition system. The 30kN pre-tension that was applied during the installation of the rockbolt was reapplied prior to subjecting the reinforced sample to a shear load.

Sample S3-2 did not fail after 88 mm of shear displacement and a maximum load of 337 kN. The test was undertaken in two stages, which involved the introduction of additional packing under the smaller specimen for additional shear displacement. A summary of the results can be seen in Table 32.
Figure 157 – Load-displacement graph for Sample S3-2 with 30kN pre-tension applied

Table 32 - Results summary for Sample S3-2

<table>
<thead>
<tr>
<th>Loading Cycle</th>
<th>Applied Shear Load</th>
<th>Axial Load at collar</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Pre-tension Applied</td>
<td>kN</td>
<td>30</td>
</tr>
<tr>
<td>Initial Shear Displacement</td>
<td>mm</td>
<td>0.0</td>
</tr>
<tr>
<td>Zone 1 Load</td>
<td>kN</td>
<td>171</td>
</tr>
<tr>
<td>Zone 1 Shear Displacement</td>
<td>mm</td>
<td>5.5</td>
</tr>
<tr>
<td>Zone 1 Stiffness</td>
<td>kN/mm</td>
<td>26.7</td>
</tr>
<tr>
<td>Peak Load</td>
<td>kN</td>
<td>337</td>
</tr>
<tr>
<td>Peak Shear Displacement</td>
<td>mm</td>
<td>47.8</td>
</tr>
<tr>
<td>Zone 2 Stiffness</td>
<td>kN/mm</td>
<td>3.1</td>
</tr>
<tr>
<td>Zero-Load Shear Displacement</td>
<td>mm</td>
<td>39.5</td>
</tr>
<tr>
<td>Loading Rate 1 (range)</td>
<td>kN/min</td>
<td>3.7-21.5</td>
</tr>
<tr>
<td>Loading Rate 2 (range)</td>
<td>mm/min</td>
<td>0.8-1.2</td>
</tr>
</tbody>
</table>

The first loading cycle clearly shows distinct loading characteristics of a reinforced sample subject to a shear load. With 30 kN of pre-tension applied to the rockbolt, the transition load reached 171 kN at a shear displacement of 5.5 mm. The peak load of 337 kN was similar to the Ultimate Tensile Strength
(UTS) of the rockbolt but the shear load was removed due to insufficient shear displacement capacity. When the load was removed, packing was inserted and the sample reached a peak load instantly of 332 kN until the load-displacement curve declined linearly to a final load of 270 kN where the applied load was removed due to insufficient shear displacement capacity.

The load cell at the collar of the borehole showed a linear increase until 125 kN where the slope of the load-displacement curve then reduced. When the reinforced sample was reapplied with a shear load the load cell increased to a maximum of 150 kN at a shear displacement of 60 mm for the duration of the test until the shear load was removed. This continuous peak load on the load cell was maintained for 30 mm of shear displacement until the load was removed.

Figure 158 shows the extent of permanent shear displacement of Sample S3-2 that can be identified by the side steel lugs that were aligned prior to applying a shear load.

Sample S3-2 was loaded at a lower loading rate compared to other samples of 12 kN per minute. The reinforcing element did not fail during this test after 88
mm of shear displacement but displayed signs of the element reaching its yield strength and approaching its UTS.

7.6.1.3. Sample S3-3

The load cell used to measure the load applied at the collar of the borehole, between the two steel plates was modified with the incorporation of a pressure transducer to enable automatic, live data readings to be acquired within the current DAQ system. The introduction of the pressure transducer within the load cell allowed accurate readings of the loads attained at the collar of the borehole and was graphed to analyse the correlation between the shear load applied to the reinforced sample and the load at the collar of the hole as shown in Figure 159.

![Analogue pressure gauge and pressure transducer incorporated with the load cell](image)

Sample S3-3 was installed with a pre-tension of 70 kN. During the initiation of testing this reinforced sample, only 42 kN was able to be applied manually onto the reinforcing element when incorporating the load cell, plates and nut.

The rockbolt failed after reaching a peak load of 398 kN and yielded with failure occurring at 369 kN at 48.6 mm of shear displacement as seen in Figure 160. A summary of the test results can be seen in Table 33.
There was a distinct change in the load-displacement curve at the transition point of 160 kN and 5.9 mm of shear displacement, indicating failure of the short length of encapsulation. The load increase up to this transition point had a
stiffness of 27.1 kN/mm and reduced to 6.3 kN/mm until the peak load of 398 kN was attained at 42.9 mm of shear displacement.

The load cell at the collar of the borehole showed two distinct points where the slope of the load-displacement curved changed first at 53 kN (7.9 mm) and finally at 152 kN (30.6 mm). The stiffness between the two transition points was 4.4 kN/mm until the load reached 161 kN and then the load was maintained at the collar of the borehole over 19 mm until the rockbolt failed.

The failure of the installed rockbolt can be seen in Figure 161 and displays the extent of breakout around the borehole and the failure mode within the failure plane of the rockbolt.

![Figure 161 – Failure of rockbolt in Sample S3-3](image)

Sample S3-3 displayed distinct changes in the load-displacement curve and can be broken into three distinct phases including the stiff initial loading, the linear loading regime until failure and the post failure period. The load cell displayed an initial linear loading rate, a stiffer loading section followed by a flat, linear loading section that remained constant at around 160 kN until failure.
7.6.1.4. Sample S3-4

Sample S3-4 was the first of the strain-gauged bolts tested (strain gauge bolt number 345). As mentioned previously, this reinforcing element was installed with no pre-tension applied and with the strain gauges orientated in a horizontal plain rather than the desired vertical plain.

The borehole was drilled 15mm deeper than originally planned. Consequently there was a shift in the location of the individual strain gauges with respect to the shear plain. The new locations are shown in Figure 162.

a) Diagram of the dimensions of Sample S3-4
b) Diagram showing the location of the parts of strain gauges along the rockbolt for Sample S3-4 measured from the shear plane

Figure 162 – Schematic diagrams showing the location of the strain gauges in Sample S3-4

The load cell was installed at the end of the rockbolt as shown in Figure 163. It was generated as a result of the rockbolt being constrained by the nut and bearing plate.
With no pre-tension applied to the reinforcing element the load-displacement curve (shown in Figure 164) was similar to previous tests. The initial loading of the reinforced sample had a stiffness of 18.1 kN/mm until reaching the transition point at 110 kN and 11 mm shear displacement. The peak load of 342 kN was obtained after 49.8 mm of shear displacement, corresponding to a stiffness of 7.7 kN/mm. Figure 164 also shows curves for the strain gauge values and load measured by the load cell.

The curve for the load cell located at the collar of the borehole had two distinct transition points at 47 kN and 135 kN followed by the load remaining constant at 149 kN until the rockbolt failed. Between the two transition points the load-displacement curve indicated a stiffness of 6.0 kN/mm. A summary of the test results can be seen in Table 34.
Figure 164 – Strain and load-displacement graph for Sample S3-4 with 3kN pre-tension applied

Table 34 - Results summary for Sample S3-4

<table>
<thead>
<tr>
<th>Loading Cycle</th>
<th>Applied Shear Load</th>
<th>Axial Load at collar</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pre-tension Applied</th>
<th>kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Initial Shear Displacement</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Zone 1 Load</th>
<th>kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>110</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Zone 1 Shear Displacement</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.0</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Zone 1 Stiffness</th>
<th>kN/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.1</td>
<td>2.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Peak Load</th>
<th>kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>342</td>
<td>149</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Peak Shear Displacement</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>49.8</td>
<td>52.9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Zone 2 Stiffness</th>
<th>kN/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.7</td>
<td>6.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Zero-Load Shear Displacement</th>
<th>mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Loading Rate 1 (range)</th>
<th>kN/min</th>
</tr>
</thead>
<tbody>
<tr>
<td>58.6-69.5</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Loading Rate 2 (range)</th>
<th>mm/min</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.4-9.1</td>
<td></td>
</tr>
</tbody>
</table>

Sample S3-4 failed in a similar manner as previous rockbolts. It was because of the orientation of the strain gauges in the horizontal direction that Sample S3-4 failed in tension. The failure was initiated at the fibrous zone in the centre of the
rock, radiating out from this zone and finally failing through the development of the shear-lip zone as seen in Figure 165.

Figure 165 – Failure of rockbolt in Sample S3-4

Figure 166 shows the orientation of the strain gauges in the rockbolt and the extent of crushing in the concrete. The relatively strong concrete minimised the extent of crushing around the borehole with Figure 166 showing the interaction between the rockbolt, encapsulation medium and concrete rockmass.

Figure 166 – Side profile of the failed rockbolt in Sample S3-4
7.6.1.5. **Sample S3-5**

The strain-gauged rockbolt (number 347) was installed with the strain gauges orientated in the vertical plain as required to analyse the stress distribution in the upper and lower most portions of the rockbolt (i.e. where stress was at a maximum and minimum).

The borehole was drilled short of the required length leading to a misalignment with the shear plain. Figure 167 shows the location of the strain gauge pairs relative to the shear plain.

![Diagram of the dimensions of the installed reinforced Sample S3-5 and the position of the reinforcing element in the concrete](image)

a) Diagram of the dimensions of the installed reinforced Sample S3-5 and the position of the reinforcing element in the concrete
b) Diagram of the location of the strain gauges in the rockbolt installed in Sample S3-5 and the distances away from the shear plane

Figure 167 – Diagram showing the location of the strain gauges in Sample S3-5

To measure the longitudinal displacement during the shear testing of the reinforced sample, an LVDT was placed at the free-end of the rockbolt as shown in Figure 168. The longitudinal movement was difficult to visually inspect as this displacement was quite small throughout the testing procedure.

Figure 168 – Location of LVDT to measure longitudinal displacement during the testing process
A new LVDT with a longer measuring stroke of 100mm was used to measure shear displacement as shown in Figure 169. Previously LVDTs would have to be reset once the shear displacement approached 50 mm but with the increased stroke length this issue was removed.

![Figure 169 – Location of new LVDT used to measure shear displacement](image)

As shown in Figure 170 the system was initially very stiff as would be expected with the locking up of the two blocks due to pre-tension. The stiffness was 130 kN/mm up to approximately 50 kN. This reduced to 23 kN/mm up to 155 kN and then averaged 6 kN/mm up to the failure load at 326 kN. The overall displacement to failure was 43.3 mm. The loading rate was similar to previous tests varying in the range of 38 to 56 kN/min. A summary of the results can be seen in Table 35.
Figure 170 – Strain and load-displacement graph for Sample S3-5 with 55kN pre-tension applied

Table 35 - Results summary for Sample S3-5

<table>
<thead>
<tr>
<th></th>
<th>Applied Shear Load</th>
<th>Axial Load at collar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loading Cycle</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Pre-tension Applied</td>
<td>kN 54</td>
<td></td>
</tr>
<tr>
<td>Initial Shear Displacement</td>
<td>mm 0.0</td>
<td></td>
</tr>
<tr>
<td>Zone 1 Load</td>
<td>kN 155</td>
<td>83</td>
</tr>
<tr>
<td>Zone 1 Shear Displacement</td>
<td>mm 5.5</td>
<td>13.3</td>
</tr>
<tr>
<td>Zone 1 Stiffness</td>
<td>kN/mm 22.6</td>
<td>2.2</td>
</tr>
<tr>
<td>Peak Load</td>
<td>kN 326</td>
<td>150</td>
</tr>
<tr>
<td>Peak Shear Displacement</td>
<td>mm 43.3</td>
<td>43.3</td>
</tr>
<tr>
<td>Zone 2 Stiffness</td>
<td>kN/mm 6.4</td>
<td>3.5</td>
</tr>
<tr>
<td>Zero-Load Shear Displacement</td>
<td>mm N/A</td>
<td></td>
</tr>
<tr>
<td>Loading Rate 1 (range)</td>
<td>kN/min 37.8-55.7</td>
<td></td>
</tr>
<tr>
<td>Loading Rate 2 (range)</td>
<td>mm/min 2.5-6.0</td>
<td></td>
</tr>
</tbody>
</table>
The LVDT that was located at the free-end of the rock bolt did not register any axial displacement. This was probably due to friction between the LVDT and steel plate. The slow loading rates allowed the tip of the LVDT to be carried upwards with the smaller specimen. A frictionless surface is required.

Figure 171 shows the failed surface of the strain-gauged rockbolt.

![Figure 171 – Failure of strain-gauged rockbolt in large specimen of Sample S3-5](image)

The rockbolt failed within the plastic hinges that are formed during the shearing process instead of between the plastic hinges as in the previous tests. An illustration of the failure location of Sample S3-5 compared to the previous samples can be seen in Figure 172. The rockbolt failed offset the shear plane and consequently a protruding failed rockbolt was observed in the smaller specimen as seen in Figure 173. The failure surface of the rockbolt was observed a minor distance into the larger specimen from the shear plane.
The failure of the strain-gauged rockbolt occurred in the zone where there was maximum tensile stress within the plastic hinges that are created during the shearing process. Figure 174 shows where the failure was initiated in the upper section of the rockbolt adjacent to the machined slot for the strain gauges and the data wires. This failure or fibrous zone quickly extended through the section of the rockbolt creating a prominent radial zone and finally a shear-lip at the base of the cross-sectional rockbolt.
Failure was due to the presence of the slot in the rockbolt and its proximity to the zone of maximum bending stress.

7.6.1.6. Sample S3-6

Sample S3-6 was the final reinforced sample (strain-gauged rockbolt # 346) to be tested in the Stage 3 testing program. The rockbolt was installed with a similar pre-tension to Sample S3-5 of 55 kN. A pre-tension of 38 kN was attained when the face plate, nut and load cell were reinstalled prior to testing, due to limitations in manually torqueing the rockbolt insitu in the laboratory.

A LVDT was again placed at the free-end of the reinforced sample to measure the axial displacement of the smaller specimen as shown in Figure 175.
The positions of the strain gauges in the rockbolt could not be accurately determined. From visual observations and it was assumed that the strain gauges were positioned symmetrically either side of the shear plane as designed.

Starting from the pre-tension load of 38 kN applied to the reinforced element, the peak load measured at the collar of the borehole reached 164 kN. The axial load-displacement curve for the load cell placed between the face plate and concrete surface gradually increased from the initial pre-tension load of 38kN to 51 kN after which the gradient of the load-displacement curve increased. The stiffness of the load cell load-displacement curve was 3.0 kN/mm until the curve started to flatten out from 157 kN until failure of the rockbolt at 164 kN.

The applied shear load reached a peak load of 281 kN. The load-displacement curve for Sample S3-6 was similar to Sample S3-5. Once the peak load was reached at 45 mm of shear displacement, there was evidence of the rockbolt yielding until failure at 56 mm of shear displacement as seen in Figure 176. A summary of the results can also be seen in Table 36.
Figure 176 – Strain and load-displacement graph for Sample S3-6 with 38kN pre-tension applied

Table 36 - Results summary for Sample S3-6

<table>
<thead>
<tr>
<th>Loading Cycle</th>
<th>Applied Shear Load (kN)</th>
<th>Axial Load at collar (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Pre-tension Applied</td>
<td>kN</td>
<td>38</td>
</tr>
<tr>
<td>Initial Shear Displacement</td>
<td>mm</td>
<td>0.0</td>
</tr>
<tr>
<td>Zone 1 Load</td>
<td>kN</td>
<td>85</td>
</tr>
<tr>
<td>Zone 1 Shear Displacement</td>
<td>mm</td>
<td>4.2</td>
</tr>
<tr>
<td>Zone 1 Stiffness</td>
<td>kN/mm</td>
<td>14.8</td>
</tr>
<tr>
<td>Peak Load</td>
<td>kN</td>
<td>281</td>
</tr>
<tr>
<td>Peak Shear Displacement</td>
<td>mm</td>
<td>45.0</td>
</tr>
<tr>
<td>Zone 2 Stiffness</td>
<td>kN/mm</td>
<td>4.8</td>
</tr>
<tr>
<td>Zero-Load Shear Displacement</td>
<td>mm</td>
<td>N/A</td>
</tr>
<tr>
<td>Loading Rate 1 (range)</td>
<td>kN/min</td>
<td>7.5-11.4</td>
</tr>
<tr>
<td>Loading Rate 2 (range)</td>
<td>mm/min</td>
<td>0.8-1.6</td>
</tr>
</tbody>
</table>
Sample S3-6 was loaded at a reduced loading rate of 7.5 to 11.4 kN/min, or 0.8 to 1.6 mm/min.

The strain-gauged rockbolt in Sample S3-6 failed in a similar manner as Sample S3-5. The rockbolt failed at the point of greatest bending moment, which was at the point where the plastic hinges formed. The rockbolt in the smaller specimen protrudes as in the previous test as seen in Figure 174. The failure surface was located a small distance into the large concrete specimen.

Figure 177 – Side profile of failed rockbolt within the smaller specimen of Sample S3-6

Failure was initiated at the corner edges of the slot indent in the rockbolt in the plane of maximum shear stress. There was minimal crushing evident around the borehole due to the relatively high strength concrete used to simulate the surround rockmass. There was evidence of cracking extending to the perimeter of the concrete block in the same direction as the applied shear load as seen in Figure 178.
7.6.2. Summary

From the Stage 3 testing program, it can be concluded that the modifications in the design of the shear testing facility have improved the ability to test the shear performance of reinforced samples. The modifications to the shear testing facility within the Stage 3 testing included an upgrade to the DAQ system, which enabled the recording of 16 additional strain gauges. A LVDT with a longer stroke length of 100 mm enabled the full testing cycle to be recorded.

The lower steel block was modified to increase the shear displacement capacity in the initial loading cycle. A section was removed from the vertical support beams to allow increased access to the free end of the reinforcing element system, necessary to connect the data cables to the strain-gauged rockbolts.

Installation of the rockbolts with the roofbolter lead to some issues which principally affected the consistency between each reinforced sample with respect to borehole diameter, borehole length, borehole orientation and most importantly pre-tension applied to the rockbolt. When applying a pre-tension to the rockbolt, a set pre-tension was required to gain consistency and
repeatability during the testing program. Attempting this level of pre-tension was difficult due to the limited torque capacity of the roofbolts and the majority of the pre-tension had to be applied manually. When applying a pre-tension within the strain-gauged rockbolts, the final orientation of the stain gauges was to be in a vertical plain. The first strain-gauged rockbolt was installed with the strain gauges aligned in a horizontal plane through two subsequent strain-gauged rockbolts in a near vertical orientation.

The first three standard JBX rockbolts were installed as per the Stage 2 tests. Sample S3-1 was over drilled and consequently no pre-tension was applied to the rockbolt. Samples S3-2 and S3-3 were installed with a pre-tension of 30 kN and 70 kN respectively. The first of the strain-gauged reinforced samples, S3-4 was installed with no pre-tension and the following two strain-gauged reinforced samples, S3-5 and S3-6, were installed with 54 kN and 38 kN respectively.

A summary of the test results can be seen in Table 37:

<table>
<thead>
<tr>
<th>Sample</th>
<th>Pre-tension applied</th>
<th>Peak shear load</th>
<th>Peak axial load at collar</th>
<th>Peak shear displacement</th>
<th>Reinforcing element failed</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-1</td>
<td>0</td>
<td>166</td>
<td>N/A</td>
<td>49.5</td>
<td>No</td>
</tr>
<tr>
<td>S3-2</td>
<td>30</td>
<td>337</td>
<td>150</td>
<td>47.8</td>
<td>No</td>
</tr>
<tr>
<td>S3-3</td>
<td>42</td>
<td>397</td>
<td>161</td>
<td>42.9</td>
<td>Yes</td>
</tr>
<tr>
<td>S3-4</td>
<td>3</td>
<td>342</td>
<td>148</td>
<td>49.8</td>
<td>Yes</td>
</tr>
<tr>
<td>S3-5</td>
<td>54</td>
<td>326</td>
<td>150</td>
<td>43.5</td>
<td>Yes</td>
</tr>
<tr>
<td>S3-6</td>
<td>38</td>
<td>281</td>
<td>164</td>
<td>45.0</td>
<td>Yes</td>
</tr>
</tbody>
</table>
From the results in Table 37, the maximum peak applied shear load was 397 kN with Sample S3-3. Samples S3-2 and S3-2 did not fail during testing. No face plate and nut was installed on Sample S3-1, which provided no confinement pressure to the rockbolt during the shearing process and allowed the rockbolt to move longitudinally once the applied shear stress was greater that the confining encapsulation stress.

The concrete had a relatively high mean strength of 70.5 MPa, Young’s modulus of 32.7 MPa and Poisson’s ratio of 0.13. There was minimal breakout observed around the collar of the borehole with the higher strength concrete.

There were issues with the strain-gauged rockbolts not being accurately installed as planned in the original design. The strain gauges were located at select distanced along the rockbolt starting at 12.5 mm from the shear plane. Due to the inability to control the installation environment, the boreholes were drilled to incorrect lengths. This subsequently led to miss-alignment of the strain gauges from the vertical plane and parallel to the shear plane.

The strain-gauged Sample S3-4 was installed with the strain gauges orientated horizontally. This rockbolt failed at a lower load compared to the standard JBX rockbolt but at a higher load than the subsequent strain-gauged samples that were installed with their strain gauges in vertical direction.

Failure of the strain-gauged samples S3-5 and S3-6 was initiated within the zone of maximum bending moment, within one of the plastic hinges that were created during the shearing of the rockbolt. Failure originated in the corner of the slot that runs along the length of the rockbolt to allow for the installation of the strain gauges and data wires. This weakness plane intersected the area of maximum stress and failure was observed close to the shear plane, compared to the previous tests which failed between the plastic hinges within the shear plane.
The strain gauge readings were volatile and produced no meaningful quantitative results. Interpreting the strain gauge results in relation to shear displacement produced strains in excess of 15,000,000 micro strain. In the strain-gauged rockbolt calibrations approximately 500 micro strain was equivalent to 100 kN of axial load. This discrepancy in magnitude between the calibration results and actual testing was likely to be due to problems with the DAQ hardware recording system.

Despite this, the correlation between the transition points in strain levels between shear load and shear displacement were visually interpreted to analyse at what position on the load-displacement curve the strain in the rockbolt changed and how this strain was maintained or varied within the rockbolt through the test.

7.7. Conclusions

The maximum shear load attained when the end of the rockbolt was not constrained with a face plate and nut was 166 kN. This applied load was the load at which the resin encapsulation failed due to the increased shear stress along the resin and concrete rockmass interface. The hydraulic load cell installed at the collar of the borehole for Sample S3-2 and subsequent tests recorded maximum axial load values between 150 and 164 kN, which can be correlated with the load that the resin encapsulation started to fail and this maximum load was maintained linearly over the duration of the test until ultimate failure of the rockbolt.

The introduction of strain-gauged rockbolts to analyse the strain along the rockbolt on either side of the shear plane changed the failure mode of the rockbolts. The slot created in the rockbolt to accommodate the gauges created a weakness point and subsequently intersected the zone of maximum bending stress leading to failure initiating at this point at relatively low loads compared to the standard JBX rockbolt. This failure point was located within one of the
plastic hinges that were created during the shearing process at the zone of maximum bending moment.

With a higher pre-tension applied to the reinforcing element, the load-displacement curve of the rockbolt under applied shear developed a higher stiffness until reaching a transition point on curve. The transition point was where the curve gradient decreases and maintains a consistent residual level of reduced shear stiffness until failure of the rockbolt. Results indicate that this transition point was attained at a much lower shear displacement when an increased pre-tension was applied to the rockbolt, i.e. higher initial rockbolt shear stiffness due to pre-tension. The stiffness post transition point was similar for reinforced samples with or without pre-tension applied.

The strain-gauged rockbolts indicated a dramatic increase of a positive axial load within the reinforcing element, which was maintained consistently through the element until failure. Due to minor issues with the installation and positioning of the strain-gauged rockbolts the results were inconclusive, in regard to load distribution along the bolt. The quantitative strain values were also inconclusive due to problems with the DAQ hardware recording system.

The shear load at failure was significantly higher than the load required to exceed the direct shear strength of the steel in the rockbolt (typically 50% of the tensile strength). This type of behaviour is very dependent on both the strength and stiffness of the surrounding rock or concrete material.
8. Failure of Reinforcing Elements

All the reinforcing elements that failed due to an applied shear load did so in a ductile manner. This final ductile failure of the steel occurred between the two plastic hinges (bending regions) that were formed due to the shear displacement of the two concrete blocks (where bending moment was greatest). Between these two hinges, the reinforcing element was subject to an axial load causing the element to fail axially in tension (hence the element failing at shear loads well in excess of the steel shear strength). This typical failure and necking of the element can be seen in Figure 179.

![Figure 179](image)

**Figure 179** – Side profile of a typically failed reinforcing element subjected to a shear load

Inspection of the failed surface of the reinforcing element confirmed the element failed mechanism as being a typical ductile bending, necking and tensile failure. The failure initiated in the centre of the necked region with the crack, then progressed laterally towards the edge of element in the area known as the radial zone. The fracture was completed via a shear lip on the outer extremities of the element. A reinforcing element that was subject to a pure axial load creates a symmetrical shear lip around the outer edge of the failed steel section, whereas the shear lip in the failed element subject to a shear load created a more ellipsoidal shape, engaging at the upper and lower section of the
element as seen in Figure 180. There shear lip was negligible at the sides of the element where the applied shear load was perpendicular to the element.

![Figure 180](image)

**Figure 180** – Typical end profile of a failed reinforcing element subjected to a shear load

The development of this unique shear lip can be due to the final rupture of the element occurring at the ends where maximum stress was located in this section of the element. When a shear load was applied to the element, the greatest stress within the element was located in the same plane as the applied load where the element was subjected to a tensile and/or compressive stress at either extremity. This final rupture of the element due to the shear lip occurred predominately in the same plane where the shear load was applied, compared to the uniform smooth annular area formed adjacent to the free-surface of the element when subjected to a pure axial load.

Figure 181 displays similar failure characteristics to Figure 180 but with an irregular final failure surface. This reinforcing element failed from a common fracture surface that radiated out until the creation of the shear lip. The shear lip that was created was very irregular due to the shear load that was continually applied upon the element while also failing axially. Even though the final failure of the steel was in a ductile manner, the final characteristics etched within the element was influenced by the applied shear load especially with the final failure
mode of the steel from the radial zone to the shear lip zone and then to the free surface of the element.

Figure 181 – Alternative end profile of a failed reinforcing element subjected to a shear load

To further analyse the failure mechanisms within the reinforcing element, scanning electron microscope (SEM) analysis was undertaken of the fracture surface of the failed element in Figure 181, with the approximate location of the photos of the fracture surfaces shown below in Figure 182. The SEM photos were taken at the School of Materials Science and Engineering at the University of New South Wales.

Figure 182 – Location of the photos taken with the SEM on the failure surface of a failed reinforcing element
RB1 was located at the flat face fracture and RB2 was located on the shear lip, which was also referred to as the cup-and-cone fracture. Both the SEM photos can be seen below in Figure 183 and Figure 184.

Figure 183 – Section RB1 SEM photo of failed surface of reinforcing element

Figure 184 – Section RB2 SEM photo of failed surface of reinforcing element
The SEM results indicated the phenomenon of a dimpled rupture, which occurred via the process of microvoid coalescence. The two fractures started in the centre of the section of the reinforcing element and then radiated outward. Once the crack was near the surface the stress state changed from triaxial to plane strain and this was responsible for the change from flat face fracture that was perpendicular to the tensile axis as shown in Figure 183 to slant fracture (45 degrees to the tensile axis) that produces the shear lip as shown in Figure 184 (Crosky, 2005).

The strain-gauged rockbolts failed uniquely when compared to the standard rockbolts due to the slot that runs the entire length of the rockbolt that allows for the insertion of the strain gauges and data wires. When the strain gauges are orientated in a direction that was aligned with the shear plane, failure of the rockbolt initiates from one of the slots at a location where there was maximum bending stress. This area was where the plastic hinges are created once a shear load was applied to the reinforced sample and leads to the rockbolt failing offset to the shear plane and consequently a protruding failed rockbolt was observed in the smaller specimen as shown in Figure 185. The failure surface of the rockbolt was then consequently observed a minor distance into the larger specimen from the shear plane.

Figure 185 – Side profile of a failed strain-gauged rockbolt within the smaller specimen
The failure of the strain-gauged rockbolt occurred in the zone where there was maximum tensile stress within the plastic hinges created during the shearing process. Figure 186 shows the location of where the failure was initiated in the upper section of the rockbolt (where the rockbolt was slotted out to allow the installation and positioning of the strain gauges and the data wires). This weakness point in the rockbolt was orientated to where the maximum tensile stress and bending moment was observed during the shearing of the rockbolt. This failure or fibrous zone quickly displaces through the section of the rockbolt creating a prominent radial zone and finally a shear-lip at the base of the cross-sectional rockbolt.

Figure 186 – Surface failure profile of Sample S3-5

This unique failure was due to the slotted section of the rockbolt and the location of this slot within the zone of maximum bending stress.
9. Research Conclusions and Recommendations

There were numerous changes to the original design of the shear testing facility, which initiated from sketches, ideas and consulting industry/technical personnel. Even once the testing facility was fabricated and assembled, modifications to the facility occurred after and during the different stages of testing. These modifications and enhancements enabled the testing program to be improved in quality and potential for testing the shear performance of installed rock reinforcement elements.

The major modifications to the shear testing facility included increasing the maximum allowable shear displacement than what was originally designed for and incrementally adjusting and improving the Data Acquisition System (DAQ) to allow for increased displacement, pressure and strain gauge readings.

The objective of this project was to design, construct and commission a full-scale laboratory shear testing facility that replicates the influence of shear forces on installed rock reinforcing elements that are present in the underground environment. The single failure plane design adopted in the test rig has been successful in allowing shear loading to be directly applied to fully installed (and where relevant, pre-tensioned) reinforcing elements.

Previous research on the shear performance of reinforcing elements and the design of laboratory testing facilities were analysed to understand the strengths and weaknesses of each unique facility and understand the strengths and weaknesses from each research project. Over the past 30 years of testing and research, there is a general agreement between in regards to the effects of specific parameters that affect the shear performance of rock reinforcement elements. There seems to be a lack of understanding of the actual reinforcement installation method and the associated loading mechanisms that may cause failure.
There was evidence of rotation of the two specimens during the tests, but this was minimal when the blocks were symmetrically aligned. A normal force was subjected to the two specimens which was seen by the formation of friction marks when the smaller specimen was displaced past the larger specimen. The amount of rotation or the assessment of the normal force generated was not calculated but visually interpreted and recorded.

The various conclusions are as follows:

- Standard rock bolts installed in the concrete rock mass can offer a shear resistance more than double the shear strength of the steel, and in fact also higher than the ultimate tensile strength (UTS) of the rock bolt steel itself due to the friction between the two specimen surfaces. There are two distinct loading characteristic stages generated from the applied shear load and shear displacement curve. The initial loading regime was much stiffer until reaching a transition point, beyond which the load-displacement stiffness was reduced and maintained until failure of the rock bolt. Even upon removing the applied shear load and reapplying a shear load, the subsequent load-displacement curves follow this same loading regime.

- At relatively higher loading rates the stiffness of the shear load-displacement curves was much higher compared to the tests with a relatively lower applied loading rate.

- A relative stiffer and stronger rock mass material caused the rock bolt to fail within a lower shear displacement compared to a relatively softer and weaker material. This was due to the reduced bolt length “activation zone” in the stronger rock mass with minimal crushing around the extent of the borehole compared to the weaker rock mass.

- When there was no confining pressure from a face plate and nut at the collar of the borehole, the rock bolt was slowly pulled through the borehole at a
constant applied load, which indicates a failure between the resin-rock mass interface.

- A pre-tensioned reinforcing element was shown to prevent early shear displacement at higher applied shear loads, which is beneficial in minimising initial shear movement within the surrounding rock mass. Beyond this initial loading regime the pre-tensioned element then reduces in stiffness to a consistent level until failure of the element.

- The strain-gauged rock bolts indicated a dramatic increase of a positive axial load within the reinforcing element, which was maintained consistently through the element until failure. Due to minor issues with the installation and positioning of the strain-gauged rock bolts a majority of the results were inconclusive, in regard to load distribution along the bolt.

- All the reinforcing elements failed in a typical ductile manner. The failure initiated in the centre of the necked region of the element with the crack, then progressed laterally towards the edge of the element in the area known as the radial zone. The fracture was completed via a shear lip on the outer extremities of the element.

- When the strain gauges were orientated and aligned with the shear plane, failure of the rock bolt was initiated from one of the slots at a location where there was maximum bending stress.

### 9.1. Recommendations

Further reporting and verification is required specifically with regard to:

- Borehole and element geometry
- Element orientation relative to discontinuity
- Element and encapsulation material geomechanical properties
Further theoretical, mechanistic and computational studies are required to support the experimental program with the final objective to prepare a set of industry guidelines for the application of the reinforcement systems in discontinuous materials, with respect to their performance in shear resistance.

Recommendations have been made to further control the laboratory experiments and minimise any variations in the results and these following recommendations are as follows:

- Due to the incorrect alignment of the strain gauges in relation of the shear plane, it is recommended that the boreholes be drilled in a controlled laboratory environment to have the boreholes of an exact length and the strain gauges align correctly to the initial custom design. This can be achieved with pre-casting the borehole with an exact length of conduit, which is positioned prior to casting the concrete and later reamed out with a drill to replicate the irregular borehole surface. The variability in the borehole lengths made the positioning and analysis of the strain gauges difficult as the strain gauges were shifted either side of the shear plane throughout the Stage 3 testing program.

- Pre-tension should be applied in a controlled laboratory environment to control the level of pre-tension applied to the reinforcing element and the orientation of the strain gauges for analysis of the stress and strain exhibited in the upper and lower sections of the reinforcing element. This can only be achieved with a mix-and-pour resin encapsulation. The reinforcing element should be secured to the base of the borehole initially to allow a pre-tension to be applied to the element and later filled with resin to secure the applied pre-tension. Using a mix-and-pour arrangement can introduce issues relating
to air pockets in the resin and method of encapsulating, but these issues can be controlled with correct installation procedures.

- The DAQ system must be upgraded to allow for accurate recordings of the strain gauges. The DAQ system was modified to allow for the provision of recording the strain gauges but due to time constraints was not fully commissioned and configured for the application within the shear test facility. There were alternative strain gauge acquiring systems available that would have allowed for the provision of strain-gauged rockbolts. These systems provide a more stringent and secure measure for acquiring and analysing data that had been purchased, but due to time constraints these systems were not utilised during the shear testing program.

- The LVDTs that were used to measure the shear displacement of the smaller specimen relative to the larger specimen were positioned using magnetic-based holders. These holders were fixed to the top reaction beams and the LVDTs positioned accordingly. The magnetic base holders could be custom made to allow for a greater securing position and remove the incorrectly sized fixing clamp, which was adjusted throughout the testing program with electrical tape.

- The LVDT that was used to measure the longitudinal displacement of the reinforcing element during the shearing process, failed to gain accurate longitudinal results due to the friction that was present between the point of the LVDT and the surface of the threaded steel plate that was used as a measuring point. The LVDT cohered to the threaded steel plate and rose with the smaller specimen instead of maintaining a fixed reference to measure the longitudinal displacement of the reinforcing element. A fixed wheel end point for the LVDT could be applied to the system or the creation of a frictionless surface between the end of the LVDT and the threaded steel plate.
• The ongoing testing program should have provision for the introduction of numerous types of reinforcing elements including cablebolts, alternative rockbolts and even ground anchors. Alternative forms of encapsulation including grouts should be examined to analyse how the different types of encapsulation can influence the overall shear performance of a reinforced sample. The reinforcement element system can be modified by changing the hole size, roughness, element diameter, element material, element mechanics and all other variances that can have an influence on the shear performance of a reinforced sample.

• In order to determine the effects of the properties of the surrounding rockmass, a testing program should be implemented to analyse the effects of the properties of the concrete rockmass on the over shear performance of a reinforced sample. Stage 2 of the shear testing program saw the use of a relatively weaker concrete to simulate the rockmass compared the concrete used in Stage 1 and 3. The installation and testing of the reinforcing elements varied between the three stages due to the fact that a face plate and nut was not installed onto the free end of the element during the Stage 2 tests. This allowed the rockbolt to shear through the encapsulation without a confining pressure applied with a face plate and nut. It would be recommended to analyse the effects of varied concrete properties and to test the reinforced samples with constant parameters and conditions.

• It has been documented that the effects of inclining the rockbolt in relation to the shear plane can have a considerable influence on the resistance to shearing of the reinforced sample. Due to the offset configuration of the larger and smaller specimens, the reinforcing element could be inclined towards the shear plane for analysis. The angle of inclination towards the shear plane can be much larger compared to the angle of inclination away from the shear plane due to the offset configuration of the shear testing facility.
• Other parameters that can have an influence on the shear performance of reinforcing elements include changing the properties of the joint surface roughness and also the joint opening/aperture.

• The mechanisms and influence of applying a pre-tension throughout the reinforcing element has to be further analysed within a controlled laboratory environment, from the casting of the concrete, drilling and installation of the reinforcing element (including applying a pre-tension) to the actual shear testing itself.
10. References

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Appendices

A1. Results Summary Tables

The Stage 2 and 3 results are summarised in the following tables with reference to the transition and peak load points indicated in Figure 187.

Figure 187 – Load-displacement curve characteristics
### Table 38 – Stage 2 Applied Shear Load Results

**Stage 2 - Applied Shear Load Results**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Loading Cycle</th>
<th>Pre-tension</th>
<th>Initial Shear Displacement</th>
<th>Zone 1 Peak Load</th>
<th>Zone 1 Shear Displacement</th>
<th>Zone 1 Stiffness</th>
<th>Peak Load</th>
<th>Peak Shear Displacement</th>
<th>Peak Stiffness</th>
<th>Zero-Load Shear Displacement</th>
<th>Loading Rate</th>
<th>Loading Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2-1</td>
<td>1-4</td>
<td>0.0</td>
<td>112</td>
<td>7.8</td>
<td>22.4</td>
<td>313</td>
<td>54.7</td>
<td>1.9</td>
<td>N/A</td>
<td>19</td>
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<td>3.6</td>
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<tr>
<td>S2-3</td>
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<td>0.0</td>
<td>139</td>
<td>10.9</td>
<td>12.4</td>
<td>226</td>
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<td>40.2</td>
<td>53 to 66</td>
<td>3.0 to 10.7</td>
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<td></td>
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<td>59 to 71</td>
<td>3.1 to 4.9</td>
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<tr>
<td></td>
<td>3</td>
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<td>361</td>
<td>76.9</td>
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<td>79.2</td>
<td>46.9</td>
<td>362</td>
<td>86.9</td>
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<td>87 to 125</td>
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<tr>
<td>S2-4</td>
<td>1</td>
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<td>131</td>
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<td>172</td>
<td>43.0</td>
<td>0.1</td>
<td>32.3</td>
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<td>0.1 to 0.5</td>
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<td>2</td>
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<td>120</td>
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<td>0.7</td>
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<td>1.5 to 7.0</td>
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<tr>
<td>S2-5</td>
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<td>204</td>
<td>11.5</td>
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<td>203</td>
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<td>64 to 77</td>
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<td>53 to 77</td>
<td>3.9 to 6.0</td>
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<td></td>
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<td>75.8</td>
<td>22.8</td>
<td>184</td>
<td>84.7</td>
<td>0.8</td>
<td>75.4</td>
<td>77 to 102</td>
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### Table 39 – Stage 3 Applied Shear Load Results

**Stage 3 - Applied Shear Load Results**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Loading Cycle</th>
<th>Pre-tension Applied</th>
<th>Initial Shear Displacement</th>
<th>Zone 1 Load</th>
<th>Zone 1 Shear Displacement</th>
<th>Zone 1 Stiffness</th>
<th>Peak Load</th>
<th>Peak Shear Displacement</th>
<th>Peak Stiffness</th>
<th>Zero-Load Shear Displacement</th>
<th>Loading Rate</th>
<th>Loading Rate</th>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>kN/mm</td>
<td></td>
<td></td>
<td>kN/min</td>
<td>mm/min</td>
</tr>
<tr>
<td>S3-1</td>
<td>0</td>
<td>0</td>
<td>110</td>
<td>8.0</td>
<td>18.1</td>
<td>166</td>
<td>49.5</td>
<td>1.2</td>
<td>-</td>
<td></td>
<td>4.2 to 11.7</td>
<td>2.0 to 9.5</td>
</tr>
<tr>
<td>S3-2</td>
<td>1</td>
<td>30</td>
<td>171</td>
<td>5.5</td>
<td>26.7</td>
<td>337</td>
<td>47.8</td>
<td>3.1</td>
<td>39.5</td>
<td></td>
<td>3.7 to 21.5</td>
<td>0.8 to 1.2</td>
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<tr>
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<td>2</td>
<td>30</td>
<td>263</td>
<td>50.3</td>
<td>39.5</td>
<td>332</td>
<td>61.3</td>
<td>-2.3</td>
<td>72.8</td>
<td></td>
<td>-7.1 to 48.7</td>
<td>1.2 to 3.1</td>
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<td>S3-3</td>
<td>42</td>
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<td>160</td>
<td>5.9</td>
<td>27.1</td>
<td>398</td>
<td>42.9</td>
<td>6.3</td>
<td>-</td>
<td></td>
<td>40.3 to 58.2</td>
<td>1.5 to 9.1</td>
</tr>
<tr>
<td>S3-4</td>
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<td>0</td>
<td>110</td>
<td>11</td>
<td>18.1</td>
<td>342</td>
<td>49.8</td>
<td>7.7</td>
<td>-</td>
<td></td>
<td>58.6 to 69.5</td>
<td>3.4 to 9.1</td>
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<tr>
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<td>155</td>
<td>5.5</td>
<td>22.6</td>
<td>326</td>
<td>43.3</td>
<td>6.4</td>
<td>-</td>
<td></td>
<td>37.8 to 55.7</td>
<td>2.5 to 6.0</td>
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<tr>
<td>S3-6</td>
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<td>85</td>
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<td>14.8</td>
<td>281</td>
<td>45.0</td>
<td>4.8</td>
<td>-</td>
<td></td>
<td>7.5 to 11.4</td>
<td>0.8 to 1.6</td>
</tr>
</tbody>
</table>
Table 40 – Stage 3 Axial Load Results

**Stage 3 - Axial Load (Collar of Borehole) Results**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Initial Load</th>
<th>Initial Shear Displacement</th>
<th>Zone 1 Load</th>
<th>Zone 1 Shear Displacement</th>
<th>Zone 1 Stiffness</th>
<th>Zone 2 Load</th>
<th>Zone 2 Shear Displacement</th>
<th>Zone 2 Stiffness</th>
<th>Final Load</th>
<th>Final Shear Displacement</th>
<th>Peak Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3-1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S3-2</td>
<td>30</td>
<td>0</td>
<td>125</td>
<td>35.1</td>
<td>2.7</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>150</td>
<td>71.7</td>
<td>150</td>
</tr>
<tr>
<td>S3-3</td>
<td>41</td>
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<td>152</td>
<td>30.6</td>
<td>4.4</td>
<td>161</td>
<td>48.6</td>
<td>164</td>
</tr>
<tr>
<td>S3-4</td>
<td>3</td>
<td>0</td>
<td>47</td>
<td>25</td>
<td>2.4</td>
<td>135</td>
<td>41.7</td>
<td>6.0</td>
<td>148</td>
<td>52.9</td>
<td>149</td>
</tr>
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<td>3.5</td>
<td>150</td>
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<td>150</td>
</tr>
<tr>
<td>S3-6</td>
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<td>0</td>
<td>51</td>
<td>9.9</td>
<td>1.3</td>
<td>157</td>
<td>44.8</td>
<td>3.0</td>
<td>164</td>
<td>53.8</td>
<td>164</td>
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</table>
## A2. Alternatives to drill and install reinforcing element

<table>
<thead>
<tr>
<th>Method 1</th>
<th>Method 2</th>
<th>Method 3</th>
<th>Method 4</th>
<th>Method 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deliver, install and commission</td>
<td>Transport specimens to Hydramatic (Newcastle) and completely drill/install bolt at site</td>
<td>Transport specimens to Hydramatic (Newcastle), completely drill and semi-install bolt on site (anchor). Pretension with a torque wrench and pour resin at laboratory</td>
<td>Cast concrete specimens with an artificial pipe to represent a smooth borehole. Install bolt into this pipe (mix-and-pour)</td>
<td>Cast concrete specimens with an artificial pipe, ream pipe with handheld drill rig to create textured surface borehole. Install bolt (mix-and-pour)</td>
</tr>
</tbody>
</table>

### Main Advantages
- Complete drilling/installation facilities at lab
- Minimum transportation/manual handling of specimens
- Best representation of actual drilling/installation of rockbolt
- Reduce lab requirements
- Space
- Support/maintenance
- Cooperation with Hydramatic
- Reduce lab requirements
- Space
- Support/maintenance
- Cooperation with Hydramatic
- Complete installation facilities at lab
- Minimal space required at lab
- Complete installation facilities at lab
- Minimal space required at lab

### Main Disadvantages
- Logistics/transportation
- Commissioning/risks
- Additional equipment
- Downtime/maintenance
- Transporting samples
- Non-lab controlled environment (loss of pre-ten when transporting)?
- Transporting samples
- Air gaps when using M&P
- Pipe not representative
- Smooth-wall borehole
- Air gaps when using M&P
- Need additional services in lab
- Air gaps when using M&P
- Need additional services in lab

### Suggested conclusions
- Not recommended
- Possible if transportation between HM and
- Possible, but influence of air gaps when using
- Not suggested due to non-representation of
- Possible with little additions to lab services but
| UNSW does not change pre-ten properties, good representation of actual installation procedures | M&P | actual borehole wall and influence of air gaps when using M&P | influence of air gaps when using M&P |
A3. Jennmar JBX Roofbolt & Steel Information
A4. LVDT Information and Electronic Setting
A5. *Hoist Beam Support Drawing*
A6. Risk Assessment

A6.1. Background

Laboratory testing will take place at the UNSW School of Mining Engineering by converting an old Avery compressive testing machine into a facility to test the shear performance of installed rockbolts.

Testing will involve installing the rockbolts into concrete moulds to replicate installation and later the shear forces that are depicted in the underground mining environment.

A rough schematic diagram of the testing facility can be seen below in Figure 188:

![Schematic of shear testing facility](image)

Figure 188 – Schematic of shear testing facility

A6.2. Expected Outcomes (& Objectives)

The objective of the risk assessment was to review the risks related to the shear testing of rockbolts installed into concrete blocks in a laboratory environment, specifically focussing on the hazards associated from gravity, mechanical,
chemical, electrical, pressure and body mechanics energies or types of problems associated with the failure of a rockbolt that was installed into concrete specimens, in order to produce specific safe working practices for this procedure.

**A6.3. Hazard & System Definitions**

The risk assessment will incorporate the outcomes from casting the specimens, drilling and installing the rockbolt, transport/positioning of specimen into testing machine, actual shear testing of the specimen and removal of specimen after testing. As mentioned in the objectives, the purpose of the risk assessment would be to create Safe Work Procedures (SWP) for each of the processes involved in the shear testing of rockbolts.

Hazards were identified through the energies that are present and examples are categorised below:

- **Gravity**
  - Specimen block falling
- **Mechanical**
  - Moving piston on testing machine
  - Projected particles (concrete, rockbolt specimens)
  - Rotating drill steels
- **Pressure**
  - Potential forces built-up in resistance of shear movement between the two blocks through the rockbolt
  - Failure of hydraulics in testing machine
- **Body Mechanics**
  - Manual handling of specimens
  - Tripping over objects in laboratory
- **Chemical**
  - Casting of concrete specimens
  - Resin encapsulating
Electrical
  o Shock, burns associated with the testing machine

A6.4. Risk Identification Method

Workplace Risk Assessment & Control (WRAC) was the appropriate risk assessment method that matched to our objectives and outcomes because WRAC enables us to:
  • List potential accidents
  • Prioritise according to risk
  • Identifies and implies the recommendation for new, potentially more effective controls, and
  • Provides a specific safe working practice (SWP).

A6.5. Risk Analysis Method

Qualitative risk analysis was used to prioritise through the likelihood versus consequence matrix and application of quantitative data was not possible in analysing the risk.

The likelihood and consequence matrix will consist of a five by five matrix and the likelihood scenarios ranging from almost certain to rare.

Consequences were based initially and most commonly on the severity in relation to injury and disease ranging from no medical treatment necessary up to a fatality or permanent disability. Secondly, consequences will also be based on damage to the testing facility, including to the actual testing machine and laboratory. The ranking of damage to the equipment and laboratory was qualitative rather than semi-quantitative as accurate figures of actual cost of damage was too limited and therefore were based on the relative cost of damage in relation to the project budget.

Effective controls and barriers would be recommended as well as the development of Safe work practice (SWP) assigned to the testing facility. These controls and SWP were vigorously stepped through and assessed in their effectiveness of reducing the proposed risks.

These SWP were reported and distributed to the risk assessment team for review and feedback.

The effectiveness of the controls/barriers and SWPs implemented would be officially witnessed and assessed through the initial use of the testing facility.
### A6.7. WRAC Risk Assessments

#### A6.7.1. Pouring concrete specimens

<table>
<thead>
<tr>
<th>No</th>
<th>Step in Operation</th>
<th>Potential Incident/Accident</th>
<th>Probability</th>
<th>Consequence</th>
<th>Risk Rank</th>
<th>Current Controls</th>
<th>Recommended Controls</th>
<th>Agreed Action</th>
<th>Accountability</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Concrete mixer entering site</td>
<td>High traffic thoroughfare, potential harm to students/staff</td>
<td>Low</td>
<td>Medium</td>
<td>5</td>
<td>Spot mixer into safe location</td>
<td>Maintain clear access for mixer</td>
<td>Spot mixer into safe location</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Moving steel casts into place</td>
<td>Manual Handling</td>
<td>Medium</td>
<td>Low</td>
<td>5</td>
<td>Have trolleys, mobile cranes handy to use</td>
<td>Ensure lifting assistance used if moving heavy loads Refer to manual handling risk assessment checklist Use mechanical aids when possible</td>
<td>Clear access in pouring area</td>
<td>Check that clear access is maintained in</td>
</tr>
<tr>
<td>3</td>
<td>Slip/trip/fall</td>
<td>Low</td>
<td>Very Low</td>
<td>3</td>
<td>Clear access in pouring area</td>
<td></td>
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</tr>
<tr>
<td>Traffic Areas</td>
<td>Health Effects from Moulding Oil</td>
<td>Lubricating Steel Cases</td>
<td>Pouring of Concrete</td>
<td>Concrete Mixer Leaving Site</td>
<td>Flying Concrete Particulates</td>
<td>Damage to Building / Truck</td>
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<td></td>
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<tr>
<td>-----------------------------------</td>
<td>---------------------------------------------------------</td>
<td>-----------------------------------------------</td>
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<td>----------------------------</td>
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<td></td>
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<tr>
<td>Low</td>
<td>Low</td>
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<td>Very Low</td>
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<tr>
<td>Ensure adequate PPE is worn</td>
<td>Have access to clean water and soap as required</td>
<td>Have access to clean water and soap as required</td>
<td>Have access to clean water and soap as required</td>
<td>Spot mixer into safe location</td>
<td>Spot mixer into safe location</td>
<td>Spot mixer into safe location</td>
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</tr>
<tr>
<td>A6.7.2. Handling concrete specimens onto transport vehicle</td>
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<table>
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<tr>
<th>Step in Operation</th>
<th>Potential Incident/Accident</th>
<th>Consequence</th>
<th>Probability</th>
<th>Recommended Controls</th>
<th>Agreed Action</th>
<th>Accountability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laying concrete sample on its side</td>
<td>Manual Handling</td>
<td>Medium</td>
<td>Medium</td>
<td>Ensure assistance is available</td>
<td>Ensure assistance is available</td>
<td>Refer to manual handling risk</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>No</th>
<th>B</th>
<th>C</th>
<th>D</th>
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</tr>
<tr>
<td><strong>Positioning concrete sample on palette</strong></td>
<td>Manual Handling</td>
<td>Medium</td>
<td>Medium</td>
<td>6</td>
<td>Guide specimen down with trolley</td>
<td>Ensure assistance is available</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Refer to manual handling risk assessment checklist</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Use mechanical aids when possible</td>
</tr>
<tr>
<td><strong>Specimen falling on self</strong></td>
<td>Low</td>
<td>Medium</td>
<td>5</td>
<td>Guide specimen down with trolley</td>
<td>Ensure steel-capped boots are worn</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>Ensure assistance is available</td>
</tr>
<tr>
<td><strong>Slip/trip/fall</strong></td>
<td>Low</td>
<td>Very Low</td>
<td>3</td>
<td>Clear access in area</td>
<td>Check that clear access is maintained around working area</td>
<td></td>
</tr>
<tr>
<td><strong>Specimen falling on self</strong></td>
<td>Low</td>
<td>Medium</td>
<td>5</td>
<td>Guide specimen down with trolley</td>
<td>Ensure steel-capped boots are worn</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Ensure assistance is available</td>
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</table>

Use mechanical aids when possible.
<table>
<thead>
<tr>
<th>Scenario</th>
<th>Priority 1</th>
<th>Priority 2</th>
<th>Action 1</th>
<th>Action 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slip/trip/fall</td>
<td>Low</td>
<td>Very Low</td>
<td>Clear access in area</td>
<td>Check that clear access is maintained around working area</td>
</tr>
<tr>
<td>Splinters / cuts / abrasions</td>
<td>Low</td>
<td>Low</td>
<td>Position hands appropriately when handling specimen</td>
<td>Wear suitable gloves</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Use mechanical aids when possible</td>
</tr>
<tr>
<td>Lifting concrete specimens onto transport vehicle with forklift</td>
<td>Low</td>
<td>Medium</td>
<td>Maintain clear access for forklift</td>
<td>Spot forklift into safe location</td>
</tr>
<tr>
<td>Damage to building / truck / forklift</td>
<td>Low</td>
<td>Medium</td>
<td>Maintain clear access for forklift</td>
<td>Spot forklift into safe location</td>
</tr>
<tr>
<td>Slip/trip/fall</td>
<td>Low</td>
<td>Very Low</td>
<td>Clear access in area</td>
<td>Check that clear access is maintained around working area</td>
</tr>
<tr>
<td>Specimen falling</td>
<td>Low</td>
<td>High</td>
<td>Keep distance away from forklift</td>
<td>Ensure assistance is available</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Ensure steel-capped boots are worn</td>
<td>Spot forklift into position</td>
</tr>
</tbody>
</table>
### A6.7.3. Setting up concrete specimens and roofbolter prior to drilling borehole

<table>
<thead>
<tr>
<th>No</th>
<th>Step in Operation</th>
<th>Potential Incident/Accident</th>
<th>Probability</th>
<th>Consequence</th>
<th>Risk Rank</th>
<th>Current Controls</th>
<th>Agreed Action</th>
<th>Accountable Controls</th>
<th>Accountable Controls</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Lifting concrete specimens into place with forklift</td>
<td>High traffic thoroughfare, potential harm to personnel</td>
<td>Low</td>
<td>Medium</td>
<td>5</td>
<td>Spot forklift into safe location</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Damage to building / truck / forklift</td>
<td>Low</td>
<td>Medium</td>
<td>5</td>
<td>Maintain clear access for forklift</td>
<td>Spot forklift into safe location</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slip/trip/fall</td>
<td>Low</td>
<td>Very Low</td>
<td>3</td>
<td>Clear access in area</td>
<td>Check that clear access is maintained around working area</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Specimen falling</td>
<td>Low</td>
<td>High</td>
<td>6</td>
<td>Keep distance away from forklift</td>
<td>Ensure assistance is available</td>
<td></td>
<td>Spot forklift into position</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Fixing concrete specimens into place prior to drilling</td>
<td>Manual Handling</td>
<td>Medium</td>
<td>Medium</td>
<td>6</td>
<td>Minimise manual handling</td>
<td>Refer to manual handling risk assessment checklist</td>
<td>Use mechanical aids when possible</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Burns from steel casing temperature</td>
<td>Very Low</td>
<td>Low</td>
<td>3</td>
<td>Check temperature of</td>
<td>Wear suitable gloves</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Task Description</td>
<td>Risk Level</td>
<td>Control Measures</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>3</td>
<td>Welding steel casings with concrete specimens into place</td>
<td>Low</td>
<td>Have clean, freshwater available and close by as required. Have welding undertaken by qualified person. Ensure adequate PPE is worn.</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Burns from welding</td>
<td>Low</td>
<td></td>
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</tr>
<tr>
<td>4</td>
<td>Positioning roofbolter into place with crane</td>
<td>Low</td>
<td>Have experienced personnel rig up and guide crane. Have clean, freshwater available and close by as required.</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Failure of hoisting equipment</td>
<td>Medium</td>
<td>Have experienced personnel rig up and guide crane.</td>
<td></td>
<td></td>
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<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Low traffic thoroughfare, potential harm to personnel</td>
<td>Very Low</td>
<td>Have experienced personnel rig up and guide crane.</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Damage to building / crane</td>
<td>Very Low</td>
<td>Have experienced personnel rig up and guide crane.</td>
<td></td>
<td></td>
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<td></td>
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</tr>
<tr>
<td></td>
<td>Instability of roofbolter</td>
<td>Very Low</td>
<td>Have experienced personnel rig up and guide crane.</td>
<td></td>
<td></td>
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<td></td>
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</tr>
<tr>
<td>5</td>
<td>Connecting power to roofbolter</td>
<td>Medium</td>
<td>Refer to manual handling risk assessment checklist. Minimise manual handling. Ensure assistance is available.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Manual Handling of cables</td>
<td>Very Low</td>
<td>Use mechanical aids when possible.</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>High</td>
<td>Have experienced personnel rig up and guide crane.</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Electrocution</td>
<td>Very Low</td>
<td>Have experienced personnel rig up and guide crane.</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td>High</td>
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</tr>
<tr>
<td>Risk Category</td>
<td>Low</td>
<td>Medium</td>
<td>High</td>
<td>Action</td>
<td>Rationale</td>
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</tr>
<tr>
<td>Connecting hydraulic hoses to roofbolter</td>
<td>Very Low</td>
<td>High</td>
<td>5</td>
<td>Ensure area is dry and free of water hazards</td>
<td>Have testing area in safe dry location</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Very Low</td>
<td>High</td>
<td>5</td>
<td>Ensures hoses are in workable condition with no tears or abrasions</td>
<td>Have experienced personnel inspect cable</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slip/trip/fall</td>
<td>Low</td>
<td>Very Low</td>
<td>3</td>
<td>Clear access in area</td>
<td>Check that clear access is maintained around working area</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 Connecting hydraulic hoses to roofbolter</td>
<td>Manual Handling of cables</td>
<td>Medium</td>
<td>Very Low</td>
<td>Minimise manual handling</td>
<td>Refer to manual handling risk assessment checklist</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Ensure assistance is available</td>
<td>Use mechanical aids when possible</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slip/trip/fall</td>
<td>Low</td>
<td>Very Low</td>
<td>3</td>
<td>Clear access in area</td>
<td>Check that clear access is maintained around working area</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Exposure to hydraulic fluid</td>
<td>Low</td>
<td>Medium</td>
<td>5</td>
<td>Ensures hoses are in workable condition with no leaks, tears or abrasions</td>
<td>Have experienced personnel inspect and connect cable</td>
<td></td>
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</tr>
</tbody>
</table>
## A6.7.4. Installing reinforcing element

<table>
<thead>
<tr>
<th>No</th>
<th>Step in Operation</th>
<th>Potential Incident/Accident</th>
<th>Probability</th>
<th>Consequence</th>
<th>Risk Rank</th>
<th>Current Controls</th>
<th>Recommended Controls</th>
<th>Agreed Action</th>
<th>Accountability</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Place drill steel into roofbolter chuck</td>
<td>Manual Handling of drill steel</td>
<td>Medium</td>
<td>Very Low</td>
<td>4</td>
<td>Minimise manual handling</td>
<td>Refer to manual handling risk assessment checklist</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Drill borehole into concrete specimen</td>
<td>Contact with rotating parts (drill steel)</td>
<td>Very Low</td>
<td>Medium</td>
<td>4</td>
<td>Keep adequate distance from rotating components</td>
<td>Ensure no free length of body (clothes, hair etc.) is in proximity to the rotating components</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fly rock from initial contact with concrete specimen</td>
<td>Very Low</td>
<td>Low</td>
<td>3</td>
<td>Keep adequate distance from drilling interface</td>
<td>Ensure adequate PPE is worn (safety glasses)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Movement of concrete specimen (if not adequately secure)</td>
<td>Very Low</td>
<td>Low</td>
<td>3</td>
<td>Keep adequate distance from drilling interface</td>
<td>Ensure concrete specimen is securely fixed prior to drilling</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>High noise</td>
<td>Medium</td>
<td>Very Low</td>
<td>4</td>
<td>Keep adequate distance from drilling interface</td>
<td>Ensure adequate PPE is worn (hearing protection)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slip/trip/fall</td>
<td>Low</td>
<td>Very Low</td>
<td>3</td>
<td>Clear access in area</td>
<td>Check that clear access is maintained around working area and water from roofbolter is drained away</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Remove drill steel from</td>
<td>Manual Handling of drill steel</td>
<td>Medium</td>
<td>Very Low</td>
<td>4</td>
<td>Minimise manual handling</td>
<td>Refer to manual handling risk</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Step</td>
<td>Task Description</td>
<td>Risk Assessment</td>
<td>Level</td>
<td>Precautions</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>4</td>
<td>Place resin cartridges into borehole</td>
<td>Contact with resin material</td>
<td>Very Low</td>
<td>Very Low</td>
<td>Inspect resin cartridges prior to placing into borehole. Have MSDS close by and readily available. Ensure adequate PPE is worn (appropriate gloves).</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Install reinforcing element into borehole</td>
<td>Contact with rotating parts (reinforcing element)</td>
<td>Very Low</td>
<td>Medium</td>
<td>Keep adequate distance from rotating components. Ensure no free length of body (clothes, hair etc.) is in proximity to the rotating components.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Contact with resin material</td>
<td></td>
<td>Very Low</td>
<td>Very Low</td>
<td>Inspect resin cartridges prior to placing into borehole. Have MSDS close by and readily available. Ensure adequate PPE is worn (appropriate gloves).</td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>
A6.7.5. **Hoisting specimen onto raised bench**

<table>
<thead>
<tr>
<th>No</th>
<th>Step in Operation</th>
<th>Potential Incident/Accident</th>
<th>Probability</th>
<th>Consequence</th>
<th>Risk Rank</th>
<th>Current Controls</th>
<th>Recommended Controls</th>
<th>Agreed Action</th>
<th>Accountability</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Ensure that specimens are secure on palette</td>
<td>Specimen falling</td>
<td>Low</td>
<td>Low</td>
<td>4</td>
<td>Keep distance away from forklift</td>
<td>Ensure steel-capped boots are worn</td>
<td>Ensure assistance is available</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Transport specimen into location</td>
<td>Slip/trip/fall</td>
<td>Low</td>
<td>Very Low</td>
<td>3</td>
<td>Clear access in area</td>
<td>Check that clear access is maintained around working</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Attach lifting slings to specimen</td>
<td>Splinters / cuts / abrasions</td>
<td>Low</td>
<td>Low</td>
<td>4</td>
<td>Position hands appropriately when handling specimen</td>
<td>Wear suitable gloves</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Slowly raise specimen</td>
<td>Specimen falling</td>
<td>Low</td>
<td>Low</td>
<td>4</td>
<td>Keep distance away from specimen</td>
<td>Ensure steel-capped boots are worn</td>
<td>Continually monitor attachments</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slings, chains or chain block falling</td>
<td>Very Low</td>
<td>Medium</td>
<td>4</td>
<td>Check integrity of slings and chains prior to attaching</td>
<td>Have experienced personnel inspect slings</td>
<td>Refer to hoist risk</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No</td>
<td>Step in Operation</td>
<td>Potential Incident/Accident</td>
<td>Probability</td>
<td>Consequence</td>
<td>Risk Rank</td>
<td>Current Controls</td>
<td>Recommended Controls</td>
<td>Agreed Action</td>
<td>Accountability</td>
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<td>----------------</td>
</tr>
<tr>
<td>1</td>
<td>Ensure that specimens are secure on palette</td>
<td>Specimen falling</td>
<td>Low</td>
<td>Low</td>
<td>4</td>
<td>Keep distance away from forklift</td>
<td>Ensure assistance is available</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Ensure steel-capped boots are worn</td>
<td>Continually monitor attachments</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Transport specimen into location</td>
<td>Slip/trip/fall</td>
<td>Low</td>
<td>Very Low</td>
<td>3</td>
<td>Clear access in area</td>
<td>Check that clear access is maintained around working</td>
<td></td>
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</tr>
</tbody>
</table>
A6.8. Risk Assessment Checklists

A6.8.1. Hoisting risk assessment checklist

This checklist was provided to assist the assessment of hoisting risks. It should be used as a guide when assessing a hoisting task. If there are any concerns after reading the checklist, the next step was to suggest control options prior to the process of risk control and contact the appropriate supervisors or personnel.

Work Location and Task

<table>
<thead>
<tr>
<th>Location</th>
<th>School of Mining Engineering main laboratory, the University of New South Wales</th>
</tr>
</thead>
<tbody>
<tr>
<td>Required Task</td>
<td>Use chain block hoist to lift test specimens from proximity location onto a raised bench</td>
</tr>
</tbody>
</table>

Key Parameters (guide only)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capacity of Hoist Support Beam</td>
<td>600 kg</td>
</tr>
<tr>
<td>Capacity of Chain Block System</td>
<td>500 kg</td>
</tr>
<tr>
<td>Capacity of 1m Flat Webbed Slings (Green)</td>
<td>2 000 kg</td>
</tr>
<tr>
<td>Capacity of 2m Flat Webbed Slings (Green)</td>
<td>2 000 kg</td>
</tr>
<tr>
<td>Capacity of 1m Endless Nylon Slings (Green)</td>
<td>2 000 kg</td>
</tr>
<tr>
<td>Capacity of 2m Endless Nylon Slings (Green)</td>
<td>2 000 kg</td>
</tr>
<tr>
<td>Weight of Complete Specimen System (Load)</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Chain Block and Hoist Risk Assessment Checklist

1. What is the capacity of the chain block and hoist?
2. How heavy is the load and where is its centre of mass?
3. Is the specimen attached it to the chain block correctly?
4. Is the bench where the specimen is to be located organised?
5. Is the path to the bench clear?

**Good practices when lifting**

- Know the weight to be lifted
- Know the capacity of the chain block and beam
- Know the capacity of the lifting tackle
- Select the lifting tackle of adequate strength and examine the equipment prior to use
- Attach the load securely to the load and hook
- Ensure that the hook is above the centre mass of the load
- Avoid shock loading of slings and chains
- Avoid using multi-leg chains and slings at an angle greater than 90° between legs
- Protect slings from sharp edges with soft packing if required

**A6.8.2. Manual handling risk assessment checklist**