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

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ABSTRACT

This paper describes the design, construction and commissioning testing program for a laboratory facility at the School of Mining Engineering, UNSW, Australia, aimed at evaluating reinforcement tendon performance when subjected to shear loading. The project has been financially supported through the coal industry ACARP research initiative. 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) rock bolts. Initial results are extremely encouraging, and illustrate a significant axial component of load development as shearing takes place – particular in lower strength, softer material types. The paper also outlines future plans for use of this facility.

INTRODUCTION

The Australian coal industry produces in excess of 350 Mtpa of black coal, resulting in over 270 Mtpa of saleable product, of which 73% is exported. This dominance of export markets results in a priority focus on consistent, reliable supply of product, together with tight constraints on mine operating costs.

Approximately 27% (94 Mtpa) of the total national production is mined from underground mines, predominantly (78%) from retreating longwall operations. There are currently 24 producing longwall mines in Australia. Typically, each of these mines would develop in excess of 10 km of development drivage per year. This longwall focus further drives the need for efficient, safe and cost-effective rapid and stable roadway drivage. Inadequate or inappropriate roof support systems can lead to unsafe roof conditions threatening both personnel safety and production capacity.

Throughout the Australian underground industry, roof support in both primary and secondary development roadways is achieved through the use of fully-encapsulated resin-anchored bolts installed as primary support, together with varying combinations of plates, straps and steel mesh, usually installed at or near the face during a cyclic in-line cut and bolt development system. This primary support is generally made up of high tensile strength X-bar 22mm diameter bolts, and is often pre-tensioned. Primary support density can vary from as few as 2 or 3 bolts per metre of roadway, to in excess of 15 bolts per metre (including rib bolts). In addition, where required (such as in major roadways and intersections, or regions of poor ground conditions or additional mining-induced stresses (such as tailgates)), secondary support is also installed in the form of cable bolts and other longer bolts and tendons, plus standing support, where necessary.

A current research project being conducted by the School of Mining Engineering at The University of New South Wales (UNSW) involves the development of laboratory facilities to evaluate the installed performance of bolts acting in shear resistance across different types of rock mass discontinuities. The effect of different levels of pre-tensioning on bolt shear performance will be included in the project testing program. This work is considered of great significance in improving the knowledge of how rock bolts perform when subjected to shear loading – a condition that is prevalent in most laminated coal mine roof strata. An initial description of the project objectives and early design concepts, together with a background review of previous similar work was provided by Hartman & Hebblewhite [1].

Rock reinforcement tendons – be they rock bolts or cables – 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. The shear resistance offered can be far greater than simply the shear strength of the tendon material (steel), since the tendons can play a major role in clamping the adjacent blocks of rock together – leading not only to mobilisation of the frictional properties of the adjacent rock surfaces, but also applying a normal force across the discontinuity to further increase the frictional resistance.

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 and quantitative significance 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 reinforcement tendon materials and installation design.

LABORATORY TESTING OBJECTIVES

The objectives of this research project are aligned within the 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.
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:
    - geomechanical properties of test block
    - element pre-tensioning
    - applied loading rate

SHEAR TESTING FACILITY DESIGN

During the initial stages of the testing facility design the key focus was on replicating an actual underground mining excavation and identifying the key features of reinforcing elements in resisting shear deformation. This facility (which incorporates the actual installation of reinforcing elements in a block or bedded rock mass) steers away from the common guillotine testing facility which can determine the direct shear strength of the tendon material, by more closely representing the interaction of the rock mass and reinforcing elements, which have a major influence on the stability of the underground environment.

An Avery-Denison compressive testing machine, previously used to test rock mass cylinders and cores (Figure 1) and capable of loads up to 3600 kN was converted into the required shear testing facility.

Located within the School of Mining Engineering at the University of New South Wales, this new testing facility provides a controlled environment to allow for future evaluations on installed reinforcing products and/ or concepts under shear loading to be undertaken.

In order to replicate a typical underground mining environment, objectives for the design of the shear testing facility included:

- 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 axial load distribution along the reinforcing element as a result of the shear load and displacement
- Determining the stress distribution around the reinforcing element (within the concrete/rock mass)

Figure 1 – Avery-Denison testing facility

In order to achieve the above objectives, two separate concrete specimens were cast and consequently coupled together through the installation of a reinforcing element. A rock mass of known material properties and strength was recreated and cast from concrete. A smooth single dominant joint surface was created during the casting of the concrete, to which the shear loads could be applied. The purpose of recreating a rock mass through casting concrete was to control the rock mass material 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 by changing the location, size, roughness and other properties of the joint.

The concrete specimens were cast in an 8mm thick steel RHS casing. The main purpose of this cast was to fully enclose the concrete specimens to prevent splitting of the concrete rock mass once a shear load was applied.

FACILITY CONSTRUCTION AND COMMISSIONING

The shear testing facility was constructed prior to the commencement of stage 1 testing. Modifications to the facility took place throughout the three testing programs. The facility's additional framework and steel casings were designed to a 600 kN capacity, well below the overall capacity of the actual Avery testing machine (3600 kN). The facility was fabricated off-site and later assembled with a final schematic of the facility shown below in Figure 2.

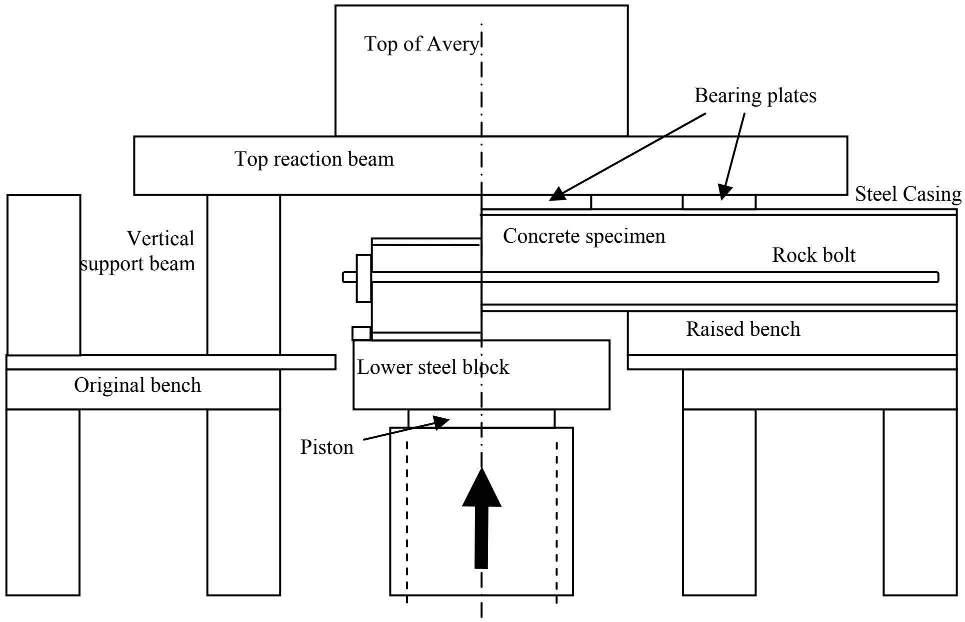


Figure 2 – Schematic of Shear Testing Facility

A data acquisition system (DAQ) was introduced within the shear testing facility. This DAQ consists of a high frequency multi-channel monitoring system capable of measuring multiple arrays of 64 channels (Figure 3).

Figure 3 – Data Acquisition System (DAQ)

The system is based on a personal computer and DASYLab software. Sampling rates can be taken at up to 1,000 readings per second with a typical experiment containing between 5,000 and 10,000 data sets. The DAQ was configured to record the voltage output of the pressure transducers from two channels and up to three channels for displacement readings from the LVDT transducers. The DAQ is able to provide the data to create load versus displacement curves (shear and axial) and allow accurate calculations of loading rates within each test.

Pressure transducers introduced within the testing facility record the pressure applied by the piston onto the reinforced sample (later in the testing program an additional pressure transducer was required to measure the axial load that was induced in the bolt between the face plate and concrete surface). This pressure transducer, based on strain gauge technology consisting of a one piece stainless steel body, has a capacity of 400 bar and provides an output of 0 to 5 volts. The hydraulic load cell (with the installed pressure transducer positioned within the two steel plates prior to testing) can be seen below in Figure 4.

The DAQ system was upgraded during the stage 3 testing program to allow for the data acquisition of strain-gauged rock bolts. Three custom strain-gauged rock bolts were manufactured specifically for this and created from the same batch of rock bolts used in the stage 2 tests.

Stage 3 testing included nine pairs of diametrically opposed strain gauges being mounted within the rock bolt to provide both axial and bending strain profiles either side of the shear plane. The locations of the stain gauges were determined to gain readings as close to the shear plane as possible. Figure displays the dimensions of the installed rock bolt in the rock mass and the locations of the strain gauges in relation to the shear plane.

Figure 4 – Application of the hydraulic load cell and pressure transducer system

Each strain-gauged rock bolt was calibrated prior to installation and testing of the reinforced sample. Even though the rock bolts had nine pairs of strain gauges along their length, the DAQ system had a limitation of only reading eight pairs at a time through two AMX interface cards. The furthest positioned pair of strain gauges was ignored due to the current limitations of the DAQ system.

The final set-up of the shear testing facility can be seen below in Figure 5. Figure 6 displays the dimensions of the installed rock bolt in the rock mass and the strain gauge locations, relative to the shear plane.

Figure 5 – Shear testing facility

PRELIMINARY TESTING PROGRAM

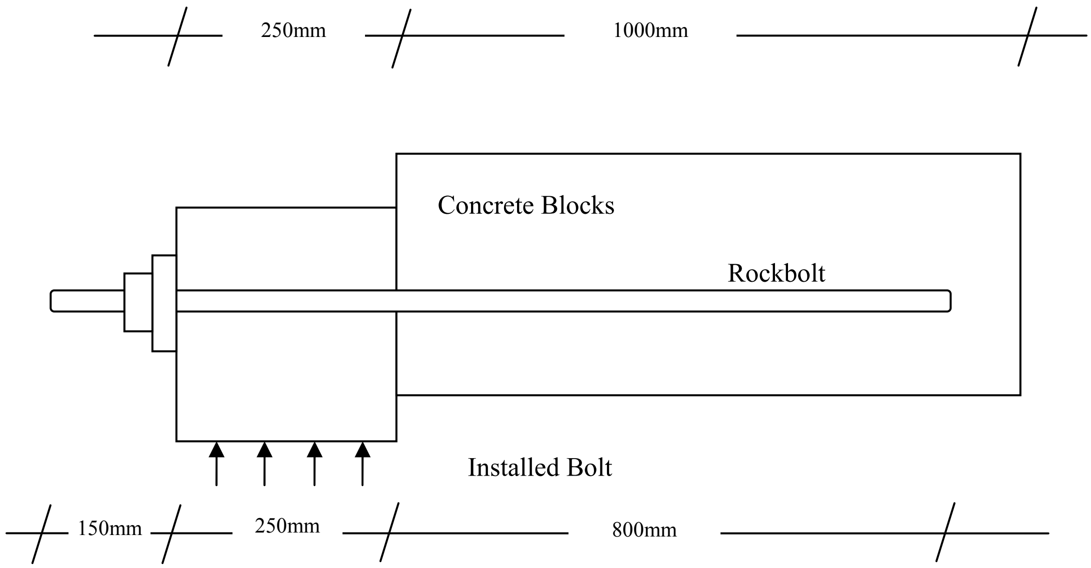
Test Program

Tests were undertaken in three stages. 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 introduced additional instrumentation and modifications to the testing facility and testing was undertaken on six reinforced samples. Stage 3 saw shear testing performed on six reinforced samples and included the installation of three strain-gauged rock bolts.

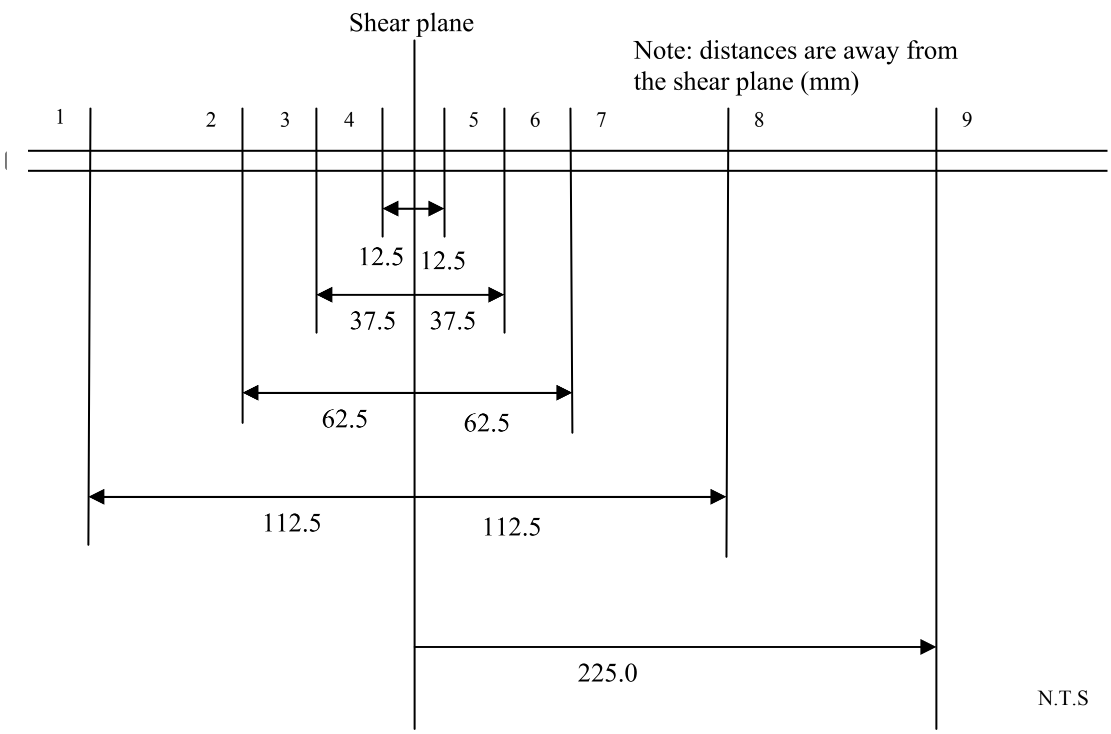
Installation of the reinforcing elements was undertaken by an ARO roof bolter, commonly used in underground coalmines throughout the world (Figure 7 and 8). Using a roof bolter to install the reinforcing elements raised minor issues in the repeatability of installation practices with variations in the borehole size and applied pre-tension across some of the reinforced samples.

Figure 7 – Drilling borehole in concrete sample 1

Figure 8 – Drilling borehole in concrete sample 2



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 rock bolt and the distances away from the shear plane

Figure 6 – Schematic diagrams showing the location of the strain gauges in the reinforcing element in relation to the shear plane

Once the samples were drilled and reinforced they were transported back to the laboratory ready for testing and analysis (Figure 9).

Figure 9 – Reinforced sample prior to testing

Initial Test Results

With no face plate installed on the reinforced sample during the testing of the stage 2 samples, each sample was loaded to a maximum of four cycles due to the maximum allowable shear displacement within the shear testing facility (40mm). The load-displacement curves that were generated during each test exhibited similar characteristics - a much stiffer initial loading phase and a distinct transition point where the stiffness of the curve dramatically decreased and remained relatively constant until the load was removed from the system. This characteristic curve was repeated in subsequent loading cycles.

Even with a faceplate installed on the bolt collars during the stage 3 testing program the load-displacement characteristics were similar across all three stages of testing. Figure 10 shows the typical load-displacement characteristics that were recorded during the shear testing process. The top line indicates the applied shear load over the duration of the test until failure, typically achieving peak shear loads of up to 400 kN. (This is more than double the shear load required to cause direct shear failure of the actual bolt steel). There is evidence of yielding, just prior to the failure of the rock bolt. The bottom line indicates the load-displacement measured by the load cell at the collar of the borehole between the faceplate and the rock mass surface (250mm from the shear plane). This result also indicates the presence of an initial pre-tension in the bolt (approx. 40kN) applied to the reinforced sample. The shearing process has clearly resulted in an increase in the axial bolt tension. The axial bolt load line trends in three phases, with the load finally remaining relatively constant at 160 kN until failure.

The reinforced samples that were not confined with a faceplate and nut did not fail within the maximum allowable shear displacement. This was due to the rock bolt slipping within the encapsulation and subsequently not having the additional confining pressure of the faceplate and nut at the borehole collar.

The reinforced samples confined with a faceplate and nut at the collar of the borehole all failed at or above the ultimate tensile strength of the rock bolt itself. Figure 11 shows the side profile of a failed rock bolt.

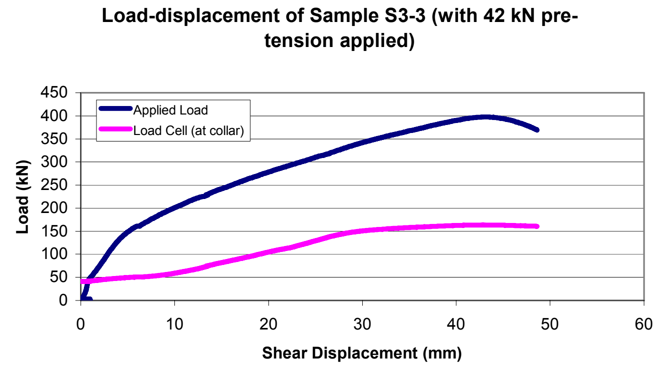


Figure 10 – Typical shear-displacement characteristics of a reinforced sample

Figure 11 – Side profile of failed rock bolt subject to an applied shear load

A preliminary series of tests were conducted using strain-gauged bolts, to assess the load distribution away from the shear plane. Unfortunately, initial problems were encountered with installation and orientation of the bolts and gauge alignment. The strain-gauged bolts were connected to the DAQ system as shown below in Figure 12.

The strain gauge readings were very volatile and did not produce reliable quantitative results within each test. Interpreting the strain gauge results in relation to shear displacement produced strains in excess of +/- 15,000,000 microstrain. The strain-gauged rock bolt calibrations only attained approximately 500 microstrain after 100 kN of applied axial load during the calibration of the rock bolts. This discrepancy in magnitude between the calibration results and actual testing results removed the validity of the strain gauge data. The problem was believed to be due to the DAQ hardware recording system. An outside source of 'noise' within the laboratory was also evident during testing, contributing to the inconclusive results.

Figure 12 – Connection of strain-gauged rock bolt to DAQ

Although the quantitative results are invalid, the correlation between strain (within the rock bolt), applied shear load and shear displacement can be visually interpreted to analyse the position on the load-displacement curve that strain in the rock bolt changes and how this strain develops within the rock bolt during the test. Figure 13 (below) also depicts the testing phase.

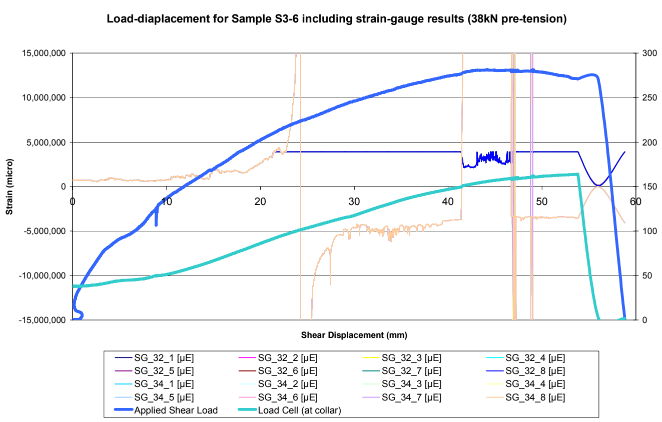


Figure 13 – Strain and load-displacement graph for sample S3-6

FAILURE OF THE REINFORCING ELEMENT

All the reinforcing elements that failed due to an applied shear load, did so in a ductile manner. This final ductile failure of the rock bolt occurred between the two plastic hinges (bending regions) that were formed due to the shear displacement of the two concrete blocks (where bending moment is greatest). Between these two hinges, the reinforcing element was subject to an axial load causing the element to fail axially, in tension (hence the bolt failing at shear loads well in excess of the steel shear strength), see Figure 14.

Figure 14 – Side profile of a typically failed reinforcing element subjected to a shear load

Inspection of the failed surface of the reinforcing element confirmed the failure mechanism as being a typical ductile bending, necking and then tensile failure. The failure initiated in the centre of the necked region with the crack, then progressed laterally towards the edge of the element in the area known as the radial zone. The fracture is then completed via a shear lip on the outer extremities of the element. A reinforcing element that is subject to a pure axial load creates a symmetrical shear lip around the outer edge of the failed rock bolt section, whereas the shear lip in the failed element subject to a shear load creates a more ellipsoid shape, engaging at the upper and lower section of the element (Figure 15). The shear lip is negligible at the sides of the element where the applied shear load is perpendicular to the element.

Figure 15 – 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 rock bolt occurring at the ends where maximum stress is located in this section of the reinforcing element. When a shear load is applied to the element, the greatest stress within the rock bolt is located in the same plane as the applied load where the element is subjected to a tensile and/or compressive stress at either extremity. This final rupture of the element due to the shear lip occurs predominately in the same plane where the shear load is applied, compared to the uniform smooth annular area formed adjacent to the free-surface of the element when subjected to a pure axial 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 by the School of Materials Science and Engineering (UNSW). The two SEM results indicated the phenomenon of a dimpled rupture, which occurs 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 is perpendicular to the tensile axis, to slant fracture (45 degrees to the tensile axis) that produces the shear lip (Crosky, 2005).

ANALYSIS OF RESULTS – INITIAL FINDINGS

The design and construction of the shear testing facility has enabled quantitative shear testing of installed reinforcing elements under controlled conditions. Numerous modifications and alterations were implemented throughout the commissioning project to achieve a testing facility to isolate and analyse the parameters that can influence the shear performance of a reinforced rock mass.

The maximum load that was attained in the reinforced samples that were not confined with a face plate and nut was approximately 160 kN. This applied load is the load at which the resin encapsulation fails due to the increased shear stress along the resin and concrete rock mass interface. The hydraulic load cell that was installed at the collar of the borehole for sample S3-2 onwards recorded maximum axial load values between 150 and 164 kN, which can also be correlated to the load that the resin encapsulation starts to fail and this maximum load is maintained linearly over the duration of the test until failure of the rock bolt.

The introduction of strain-gauged rock bolts to analyse the strain throughout the rock bolt (either side of the shear plane) changed the failure mode of the rock bolts. The slot that is created in the rock bolts to accommodate the gauges creates a weakness point and subsequently intersects the zone of maximum bending stress, initiating failure at this point at relatively low applied loads compared to the standard rock bolt. This failure point is located within one of the plastic hinges that are 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 bolt under applied shear load developed a higher stiffness until reaching a transition point on the curve. The transition point is where the curve gradient decreases and maintains a consistent residual level of reduced shear stiffness until failure of the rock bolt. Initial results indicate that this transition point was attained at a much lower shear displacement when an increase pre-tension was applied to the rock bolt, i.e. higher initial bolt shear stiffness due to pre-tension. The stiffness post transition point is similar for reinforced samples with or without pre-tension applied.

The results also confirmed that by activating a finite length of bolt either side of the shear plane to which the loading is applied, the bolt bends and ultimately fails in an axial tensile failure. The shear load at failure is significantly higher than the load required to exceed the direct shear strength of the steel in the bolt (typically about 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.

CONCLUSIONS

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 is much stiffer until reaching a transition point, beyond which the load-displacement stiffness is 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 is 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 indicate a dramatic increase of a positive axial load within the reinforcing element, which is 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 is initiated from one of the gouges at a location where there is maximum bending stress.

RECOMMENDATIONS

Further testing reporting and verification is required to fulfill the objective of this ongoing project and provide a stepping-stone for the greater objectives within the ACARP project. More definitive testing is required to study the influential parameters, including:

- 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 prevent the influence of variability in the drilling and installation process.

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