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The shear performance of cable bolts in experimental, numerical and mathematical studies

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The shear performance of cable bolts in experimental, numerical and mathematical studies

A thesis submitted in fulfilment of the requirements
for the award of the degree of

DOCTOR OF PHILOSOPHY

from

UNIVERSITY OF WOLLONGONG

by

Haleh Rasekh

B. Eng (Civil), M. Eng (Civil)

Faculty of Engineering and Information Sciences

2017
CERTIFICATION

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

Haleh Rasekh

05/2017
LIST OF PUBLICATIONS

Journal articles:


Conference articles:


Funding for this study was provided by the matching Grant Scholarship between Prof. Naj Aziz and the Faculty of Engineering and Information Sciences, the University of Wollongong.

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Last but not least, I would like to dedicate my thesis to my family, my parents and my brother, for their unconditional love and support, which enabled me to overcome all obstacles I faced during my PhD study period.
ABSTRACT

The behaviour of encapsulated cable bolts in the rock mass has been studied in the past to determine the load transfer capacity and their ultimate tensile strength. Past studies were mostly based on pull-out tests because of the ability of pull-out tests to be conducted both in the laboratory and in the field. Thus, a number of experimental and analytical models are available to determine the load transfer capacity of cable bolts mainly in pull-out conditions. On the other hand, studies on the shear performance of cable bolts are limited and are mostly based on experimental studies, which do not accurately simulate the real shear performance of cable bolts. The shear behaviour of cable bolts is significantly affected by pretension load, cable bolt surface profile type, ultimate tensile capacity, rock mass strength and cementitious grout or chemical resin properties. None of the past research studies investigated the shear behaviour of various types of fully grouted cable bolt with varied pretension loads subjected to shearing. This thesis is concerned with the study of the shear behaviour of cable bolts. Shear tests have been carried out using both single and double shear apparatus (with and without contact between concrete block surfaces).

Ten different types of cable bolts; plain and indented Superstrand, indented TG, plain and indented SUMO, twin Garford, Secura HGC, plain MW10 and spiral MW9 and plain 19 wire 21.7 mm were tested at different pretension loads, 0 t to 10 t and 25 t. Different resin and cementitious grouts were used to assess their impacts on the cable bolt shear strength. Concrete blocks with Uniaxial Compressive Strength (UCS) of 40 MPa were used in experiments to maintain consistency. Three different sets of tests were conducted to determine the shear strength and failure characteristics of various cable bolts used in Australian mines. Therefore, these three sets of test were:
- Double shear test on the pre-tensioned fully grouted cable bolts when sheared concrete blocks faces are in contact.

- Double shear test on the pre-tensioned fully grouted cable bolts when concrete blocks sheared faces are not in contact.

- Single shear test on the pre-tensioned fully grouted cable bolts when concrete blocks are not in contact.

The measured shear strength of pre-tensioned fully grouted cable bolts was found to decrease by removing friction between concrete blocks shearing joints face, thus allowing the total shear strength of the system being the full shear strength of cable bolt. The shear strength of plain surface profile cable bolt is relatively higher compared with the spiral profile cable bolt. The peak shear load and shear displacement at peak shear load decreased by increasing the pretension load. The final shear failure load appeared to be closer to failure in tension (tensile failure) rather than in shear as the axial load build up per side being nearly equal to shear failure load. It was also found that the plain wire cable strand tends to debond readily compared with the indented or rough surface wire strand cables. The single shear test was capable of evaluating the bonding and debonding of cable bolts, this being different from the double shear test where the cable was fitted with the barrel and wedge on both sides of the cable bolt. Indented and spiral cable bolts did not debond because of interlocking between the cable and the grout however, all plain surface profile cable bolts were debonded.

An analytical model, based on the energy balance theory and Fourier series concept, was developed to predict the shear strength of pre-tensioned fully grouted cable bolts under double shearing. An empirical relationship was also proposed to determine the shear strength of cable bolts subjected to single shear test. Model coefficients were
calibrated using laboratory results. In general, the model results were in good agreement with the experimental data.

The capability of Fast Lagrangian Analysis of Continua (FLAC 2D) was also investigated in simulating the shear behaviour of cable bolts subjected to double shear and single shear testing.
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LIST OF SYMBOLS AND ABBREVIATIONS

Symbols

\( A \) Outer surface area
\( A_b \) Cross-sectional area of the bolt
\( A_c \) Cross-sectional area of the cable
\( A_i \) Influencing area of the bolt in the rock
\( A_j \) Area of joint
\( A_s \) Broken area of cable in shear
\( A_t \) Broken area of cable in tension
\( a_f \) Friction length
\( a_n \) Fourier coefficient
\( a_s \) Softening length
\( a_0 \) Model Coefficient
\( b_n \) Fourier coefficient
\( b_0 \) Model coefficient
\( C \) Cohesion
\( C_C \) Contact coefficient
\( D \) Height and depth of the shear block
\( D_b \) Diameter of the bolt
\( D_c \) Diameter of the cable
\( d \) Diameter of steel bar
\( dF_p \) Incremental change in bond capacity between cable and grout
\( d_g \) Diameter of borehole
\( d_u \) Axial displacement
\( d_v \) Shear displacement
\( d_s \) Incremental frictional slip
\( d_0 \) Outside circle diameter of rock where the influence of the bolt disappears
\( E \) Modulus of Elasticity
\( E_b \) Debond’s modulus of bolt
$E_c$ Young’s modulus of cable bolts
$E_{grout}$ Young’s modulus of grout
$E_m$ Young’s modulus of the confining medium
$\Gamma M$ Penetration modulus
$E_r$ Young’s modulus of rock mass
$F_b$ Bar response to shearing
$F_g$ Global reinforced joint resistance
$F_{max}$ Ultimate tensile load of the bolt
$F_1$ Axial force acting on the bolt at strain gauge position 1
$F_2$ Axial force acting on the bolt at strain gauge position 2
f Model coefficient
$f_n$ Normal force at the contact point
G Shear Modulus
$G_G$ Shear Modulus of grout
$\Gamma G$ Modulus of rigidity
$\bar{l}$ Second moment of Inertia
i Dilation angle
$i_0$ Apparent dilation angle
K Shear stiffness
k Bulk Modulus
$K_p$ Post-peak shear stiffness of the grout/rock interface
$K_y$ Medium stiffness
$L_B$ Length of beam
$L_c$ Embedment length of cable bolt
$L_s$ Shearing length
$l$ Distance
$M$ Bending moment
$M_y$ Plastic moment
m Model Coefficient
$m_F$ Influence of rock deformability on the shear resistance of the bolted joint to the maximum tensile load of the bolt
$m_R$ Influence of the joint friction on the shearing resistance of the
bolted joint in relation to the maximum value of tensile load

\( N \)
Axial load

\( N_j \)
Frictional strength provided by the unbolted smooth joints

\( N_{oe} \)
Axial bolt force at the shear plane when the bolt yields

\( N_{of} \)
Axial bolt force acting at the shear plane during bolt failure

\( n \)
Number of Fourier coefficients

\( n_b \)
Number of bolt sections on each joint

\( n_n \)
Number of slave nodes

\( P \)
Pressure

\( P_t \)
Maximum tensile load of the bolt

\( P_u \)
Bearing capacity of rock or grout

\( p \)
Axial load corresponding to the yield strength

\( Q \)
Net energy crossing system boundary

\( Q_{cf} \)
Shear load

\( R_b \)
Radius of borehole

\( r_b \)
Radius of bolt

\( S \)
Shear force

\( S_o \)
Maximum shear resistance

\( S_b \)
Shear force carried by bolt

\( S_d \)
Dowel effect (shear force induced in the bar)

\( S_r \)
Reinforcement effect

\( S_y \)
Shear Force at the yield stress of the bolt

\( S_f \)
Shear force in bolt at the shear plane during bolt failure

\( S^* \)
Bolt contribution to shear strength

\( s \)
Frictional slip

\( T \)
Tensile force

\( T_f \)
Failure force

\( T_j \)
Joint friction

\( T_r \)
Load induced in the bar

\( T_v \)
Difference between the applied shear force

\( T_0 \)
Tension in bolt

\( t_g \)
Thickness of grout

\( u \)
Axial displacement
\( U_{ax} \)  Axial displacement of the bolt
\( U_l \)  Lateral displacement at the bolt-grout interface
\( U_{tot} \)  Total displacement
\( U_y \)  Displacement along the joint
\( u_r \)  Original radial displacement of the rock
\( x \)  Distance of any point from the steel bar’s free end
\( W \)  Net work done by or on the system
\( W_g \)  Weight of sliding mass
\( y \)  Distance from the neutral axis
\( \beta \)  Initial angle between bolt and normal to the joint
\( \beta_b \)  Bounding functions
\( \beta_r \)  Reduction coefficient of dilation angle
\( \beta_s \)  Reduction factor for the strain softening stage
\( \sigma \)  Normal stress
\( \sigma_b \)  Bond strength
\( \sigma_{b0} \)  Dilation axis by an intercept
\( \sigma_c \)  Uniaxial Compressive Strength of rock
\( \sigma_{ec} \)  Stress in bolt when bolt failure occurs
\( \sigma_g \)  Uniaxial Compressive Strength of grout
\( \sigma_{lim} \)  Limitation stress (compressive strength of grout)
\( \sigma_n \)  Effective normal stress
\( \sigma_{n0} \)  Initial normal stress
\( \sigma_0 \)  bond stress at free end
\( \sigma_y \)  Yield strength of bolt/cable bolt
\( \sigma_{b0} \)  Axial stress of the bolt at the joint
\( \tau \)  Shear stress
\( \tau_{ave} \)  Initial average bolt shear stress for impending movement
\( \tau_{crit} \)  Critical shear stress
\( \tau_0 \)  Average shear stress on the interface without a bolt
\( \tau_p \)  Peak shear stress
\( \tau_r \)  Residual shear stress
\( \tau_s \)  Residual frictional stress at the rod-grout interface
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\tau_u$</td>
<td>Shear strength at the rod-grout interface</td>
</tr>
<tr>
<td>$\tau_y$</td>
<td>Shear load at yield point</td>
</tr>
<tr>
<td>$\varphi$</td>
<td>Friction angle</td>
</tr>
<tr>
<td>$\varphi_b$</td>
<td>Concrete surface basic friction angle</td>
</tr>
<tr>
<td>$\varphi_i$</td>
<td>Friction angle of the smooth joint</td>
</tr>
<tr>
<td>$\xi_x$</td>
<td>Extension in the bolt</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Shear displacement</td>
</tr>
<tr>
<td>$\nu_g$</td>
<td>Poisson’s ratio of grout</td>
</tr>
<tr>
<td>$\nu_r$</td>
<td>Poisson’s ratio of rock mass</td>
</tr>
<tr>
<td>$\mu$</td>
<td>Coefficient of internal friction for contact surfaces</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Bolt orientation</td>
</tr>
<tr>
<td>$\theta_0$</td>
<td>Initial orientation of bolt</td>
</tr>
<tr>
<td>$\theta_1$</td>
<td>Bolt orientation at displacement $d$</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>Slip value</td>
</tr>
<tr>
<td>$\Delta T_{A+G}$</td>
<td>Expression for the bolt inclination</td>
</tr>
<tr>
<td>$\Delta \tau$</td>
<td>Shear stress at bolt-resin interface</td>
</tr>
<tr>
<td>$\Delta U$</td>
<td>Change in the internal energy of system</td>
</tr>
<tr>
<td>$\varepsilon_0$</td>
<td>Axial strain induced by the initial torque applied to the bolt</td>
</tr>
<tr>
<td>$\varepsilon_1$</td>
<td>Increment of axial strain induced by shear displacement</td>
</tr>
<tr>
<td>$\varepsilon_y$</td>
<td>Strain at the onset of yielding</td>
</tr>
<tr>
<td>$i$</td>
<td>Orientation angle of the cable bolt</td>
</tr>
<tr>
<td>$\delta$</td>
<td>Slip at the rod-grout interface</td>
</tr>
<tr>
<td>$\delta_1$</td>
<td>Slip at peak bond strength</td>
</tr>
<tr>
<td>$\delta_2$</td>
<td>Slip at bond failure</td>
</tr>
<tr>
<td>$\delta_f$</td>
<td>Opening displacement</td>
</tr>
<tr>
<td>$\delta_v$</td>
<td>Vertical displacement (Dilation)</td>
</tr>
<tr>
<td>$\delta_{max}$</td>
<td>Maximum deflection</td>
</tr>
<tr>
<td>$\delta_y$</td>
<td>Shear displacement at yield point</td>
</tr>
<tr>
<td>$\delta_p$</td>
<td>Shear displacement at peak shear load</td>
</tr>
<tr>
<td>$\epsilon$</td>
<td>Angle of stirrups</td>
</tr>
</tbody>
</table>
### Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS</td>
<td>British Standard</td>
</tr>
<tr>
<td>BSM</td>
<td>Bond Strength Model</td>
</tr>
<tr>
<td>CBG</td>
<td>Cable bolt grout</td>
</tr>
<tr>
<td>DEPT</td>
<td>Double Embedment Pull Test</td>
</tr>
<tr>
<td>FISH</td>
<td>Programming Language Embedded with FLAC</td>
</tr>
<tr>
<td>FLAC</td>
<td>Fast Lagrangian Analysis of Continua</td>
</tr>
<tr>
<td>FLAC 2D</td>
<td>ITASCA code for 2D stress analysis</td>
</tr>
<tr>
<td>ID</td>
<td>Indented</td>
</tr>
<tr>
<td>LSEPT</td>
<td>Laboratory Short Encapsulation Pull Test</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Differential Transformer</td>
</tr>
<tr>
<td>UCS</td>
<td>Uniaxial Compressive Strength</td>
</tr>
<tr>
<td>USBM</td>
<td>US Bureau of Mines</td>
</tr>
<tr>
<td>UTS</td>
<td>Ultimate Tensile strength</td>
</tr>
<tr>
<td>W:C</td>
<td>Water to cement ratio</td>
</tr>
</tbody>
</table>
Chapter I

INTRODUCTION

1.1 Rationale

Mining in Australia can be safer as there are still some fatal accidents. The number of fatalities for the past years of 2013, 2014 and 2015 was 10, 13 and 13 respectively (Safe work Australia, 2016). It is vital to decrease the number of fatal and nonfatal accidents in mines to zero. Therefore, one of the greatest concerns to designers and engineers during and after excavation is stability of underground excavations and surface mining slopes. It is important for designers to understand different forms of instability and the mechanisms of failure and associated conditions to support the unstable rock by installing various types of rock bolts and cable bolts in a proper way. The application of rebar technology is conducted in two distinct ways. Reinforcements of the immediate ground during the excavation stage with solid rock bolts are called the primary support. This type of reinforcement is aimed at strengthening the immediate layers of rock and the effectiveness of such reinforcement is dependent upon the strength and competency of the reinforced rock layers. The term secondary support is normally used to describe the additional active or passive reinforcement to guarantee stability of excavations in highly stressed or geologically disturbed strata. Usually long cable bolts are used as secondary or complementary ground supports. They are usually of longer length to enhance the existing support system around the excavation and to anchor the fractured and weak rock to more competent strata layers above. In many mines nowadays the application of secondary supports using cable bolts is a usual practice, and the significance of cable bolting is thus playing and will continue to play an important role.
for efficient and safer mining both in underground and opencut operations as well as in civil engineering endeavours.

1.2 **Background to the study**

1.2.1 **History of rock bolts**

The first use of tendons in rock reinforcement in the United Kingdom started in 1872 in a slate quarry located in North Wales (Schach et al., 1979). The use of rock bolts in a coal mine in Germany goes back to 1918, whereby the rock bolts were made of wood to prevent small rock pieces from falling into the excavation (Lang et al., 1979). In the late 1940’s, Norwegians developed rock bolting as a practical and economical technology (Bolstad and Hill, 1983).

Between 1949 and 1969, the Australian mining industry started to use rock bolting technology in the Snowy Mountains hydroelectric Scheme (Bolstad and Hill 1983). Gardner (1971) reported the use of timber support roof bolting in Australia in Elrington Colliery, New South Wales. Nowadays, roof bolting application is wide spread in Australia in almost every mine and tunnel construction. The annual usage of tendons in Australia is in the order of 7 M, consisting mainly of solid rebar bolts. Other bolts in use include split set, yieldable bolts and cable bolts. Solid bolts are normally anchored in rock using chemical resins, while cementitious grouts are largely associated with cable bolt encapsulation installation.

The installation of mechanical rock bolts in a metal mine in the United States began in 1972. The US Bureau of Mines (USBM) started roof bolting technology to decrease the number of fatal accidents in the mines. Tendon technology particularly rock bolting technology has been implemented in nearly all underground coal mines in the United States. 500 million rock bolts were installed all over the world in 2011. Among this, the
number of used bolts, cable bolts and split sets in Australia was 5,000,000, 600,000 and 250,000 respectively. 100 million roof bolts were used in the US in 2011 (Peng, 1986) and 66% of that were fully grouted resin bolts (Chao, 2012). Figure 1-1 shows the details of bolt usage in US in 2011.

![Bolt Usage US 2011](image)

**Figure 1-1** Amount of bolt usage in US in 2011 (Tadolini, 2016)
1.2.2 History of cable bolts

The application of cable bolts as a secondary support system has been a growing trend in underground coal mines worldwide (Fuller and O’Grady, 1993). The use of cable bolts as a long fully grouted steel element with high tensile strength started in the mid-1960. The Willroy mine in Canada and the Free State Geduld Mines Ltd in South Africa were the first mines to use cable bolts. The prestressed wires were installed in long flexible lengths to provide deep anchorage in the rock mass (Windsor, 1992).

The first type of cable bolts consisted of seven smooth, prestressed, high tensile 7 mm diameter wires, which were arranged with plastic spacers. The plain strand cables of poor load transfer properties due to smooth and straight profile wires were initially introduced to mines as a temporary means of rock reinforcement. Over the years, a number of modifications have been introduced to the plain strand cable, such as strands surface profiling and indentations (Schmuck, 1979), double plain strand (Matthews et al., 1983), epoxy-coated strand (Dorsten, 1984), fibreglass cable bolt (Mah, 1990), birdcage strand (Hutchins et al., 1990), bulbed strand (Garford, 1990), and nutcage strand cable bolts (Hyett et al., 1993). These various types of cable bolt have been incorporated to improve the load transfer capacity as permanent ground reinforcement.

In the early 1970’s in Broken Hill, Australia, indented strand cable bolts were introduced for the first time. This change in the profile section from plain to indented cable bolts was a great success in improving productivity, adaptability, mechanical performance and higher load transfer ability. These new types of cable bolts have the ability to be pushed into the 30 metres holes because of their rigidity. The most popular cable bolt in the world is still the 7-wire, 15.2 mm nominal diameter strand. The central strand is called as “kingwire”, which is straight and the six others are slightly smaller bordering wires (Windsor, 1992).
In the US in 2016, 1.32 million 15 mm diameter cable bolts and 25,000 18 mm vertical cable bolts were sold. The 15 mm strand cable bolt has the nominal strength of 27.22 t however the 18 mm strand cable bolt has nominal strength of 36.29 t. These two types of cable bolts are both 7 strand (King-wire and 6 outer wrap wires) (Tadolini, 2016).

In the late 1970’s and 1980’s, cable bolt technology was applied to the support and reinforcement of larger open stope spans in cut and fill mining. The reason for using cable bolts was to increase the length of support in poor ground conditions and to provide support in the early stage of mining excavations. Also, one of the main aspects of success in using cable bolts in cut and fill mining was the ability to pre-reinforcing the strata before excavation to stop any damage due to the stress re-distribution effects of mining.

Then, by 1981, the successful experiment of two revised designs including strand with additional anchors were trialled for the post-rock mass yield reinforcement of highly stressed crown pillars at Broken Hill in Australia. These anchors contained double-acting, barrel and wedge anchors or rectangular, steel swages and the intermissions between anchors were debonded by using polyethylene tubes or simply by painting them. Then, in 1983 at Mt Isa mines in Australia, the birdcage strand was introduced for cut and fill mining which had excellent load transfer characteristics. ‘Bulbed’ strand and ‘Ferruled’ strand are other types of the strands that were developed a few years later (Fuller, 1983; Windsor, 1992).

### 1.3 Key objectives

The main objective of this thesis is to investigate the performance of different types of cable bolts under various pretension loads subjected to single shear and double tests. The key objectives include:
Chapter I

Introduction

- Critical literature reviews of the previous research work in the area of cable bolt behaviour. It includes the axial and shear performance of cable bolts subjected to pull out and shear tests to investigate the load transfer performance and strength capacity of cable bolts.
- Laboratory investigation of the shear behaviour of various cable bolt surface profiles at different pretension loads using single and double shear tests.
- Study the influence of both concrete and grout (resin and cementitious) types and strengths on the cable bolt performance.
- Development of an analytical model to predict the shear performance of pre-tensioned fully grouted cable bolts in the elastic and cable induced strain softened ground at various stages of shearing.
- Development of an empirical relationship to simulate the performance of cable bolts subjected to single shear tests.

1.4 Outline of the thesis

The thesis consists of eight chapters followed by a list of references and appendices. The thesis is organised as follows:

Chapter I presents a general introduction to the present research background of this study, key objectives and the outline of the thesis.

Chapter II contains a comprehensive literature review on the history and general information about cable bolts followed by explanation of the experimental studies to study the performance of cable bolts subjected to pull testing, single shear test and
double shear test to investigate the load transfer capacity and ultimate strength of cable bolts.

Chapter III reviews the past research work on the performance of tendons including numerical and analytical simulation of tendons subjected to pull out and shear testing. This provides a better understanding on different methods of mathematical simulation.

Chapter IV presents the laboratory testing results conducted on shear behaviour of pre-tensioned fully grouted cable bolts subjected to double shear tests. The details of large scale double shear apparatus, sample preparation and an experimental plan for studying the effects of cable surface profile type, concrete blocks strength, pretension load and grout properties on shear behaviour of cable bolt are discussed. This chapter describes two series of double shear tests. The first part includes the double shear tests when the concrete block surfaces are in contact with each other while the second part includes the double shear test when the contact between concrete block surfaces are removed. Ten different types of cable bolts with different amounts of pretension loads (mostly 0, 10, 15 and 25 kN) with 100 mm shear displacement were tested.

Chapter V is devoted to examining the experimental shear test behaviour of pre-tensioned fully grouted cable bolts under single shear tests conducted. It contains the details of single shear apparatus, sample preparation and an experimental plan applied in the study. The interpretation of experimental results including shear and axial loads versus shear displacements are provided.

Chapter VI proposes an incremental analytical elasto-plastic constitutive model to simulate the shear behaviour of pre-tensioned fully grouted cable bolts in elastic, strain softening and failure stages subjected to double shear test. The model is developed by incorporating the Fourier series and energy balance theory. Moreover, a mathematical
model is also proposed to account for the shear performance of cable bolts under single shear test using the tri-linear method. The numerical simulation of the shear behaviour of pre-tensioned fully grouted cable bolts subjected to the double shear, British Standards and single shear tests in the Fast Lagrangian Analysis of Continua (FLAC) are presented to show the capability of FLAC 2D in simulation.

Chapter VII reviews the final thesis study results comparing findings from chapter IV (double shear without contact between concrete block surfaces, MKIII) and chapter V (single shear test) and also development of mathematical model proposed in chapter VI to determine its accuracy.

Chapter VIII draws together chapters II, III, IV, V and VI and highlights the key findings from the literature review and current study on cable bolts subjected to double and single shear tests. It provides a summary of the findings and conclusions of this research as well as recommendations for further studies.
LITERATURE REVIEW OF PERFORMANCE OF CABLE BOLTS-
EXPERIMENTAL STUDIES

2.1 Introduction

This chapter includes a broad description of cable bolts; principally the fully grouted cable bolts of different types used in the Australian market and review the experimental studies on the behaviour of cable bolts in axial and shear loading conditions.

Cable bolts provide the advantage of rock mass supporting itself. A cable bolt is a flexible tendon of high tensile strength strand. Each strand consists of a group of steel wires that are twisted into the strand and grouted into a borehole. The application of fully grouted cable bolts began in the underground mining industry in the 1960s. Cable bolts are installed for permanent and temporary support purposes because of their advantages in improving the stability of rock formations (Chen et al., 2016a and 2016b). The cable bolts are used in civil and mining constructions. In civil engineering constructions, they are used for tunnelling and stabilising dams and rock slopes. In both the hard rock and the coal mining industries, they are used for slope stability applications in surface mining and ground support purposes in various areas of walls, roof and floor of underground and surface openings, including: drifts, mine roadways and intersections, open stope backs, open stope walls, cut and fill stopes, draw points and permanent openings for underground mining (Ur-Rahman et al., 2015; Hutchinson and Diederichs, 1996; Windsor, 1992; Fuller, 1983; Puhakka, 1997).
1.32 million 15 mm diameter cable bolts and 25,000 18 mm vertical cable bolts were sold in the US in 2016. These two types of cable bolts are both 7 strand (King-wire and 6 outer wrap wires). Lower capacity cable bolts’ are used in USA and Canada for cost cutting purpose and lower capacity cable bolts are efficient in most cases (Tadolini, 2016).

2.2 Cable bolts background

Cable bolts are a multi-purpose type of support with the following advantages (Hutchinson and Diederichs, 1996). They:

- Make a safe working environment,
- Increases the stability of rock mass,
- Control the dilation of waste rock from the stope borders,
- Curve around constricted radii,
- Make installation of flexible long bolts possible from limited working places,
- Provide a variety of performance characteristics due to different steel wires configurations, and
- Provide general strata reinforcement in the vicinity of longwalls in coal mining.

And their:

- Capability to install more than one cable bolt strand in a big single borehole to raise tensile capacity,
- Ability to be restrained by attaching plates, straps and mesh, and
- Capability of installation in a group with other support systems such as shotcrete, mechanical bolts or grouted rebar.
There are different methodologies for installation of cable bolts. The first method is to install the cable bolt in a hole, grout the anchorage at the far end, fit face restraint, prestress the cable bolt, and grout the hole. Another methodology is to first prestress the cable bolt and then grout the cable bolt. Pre or post-reinforcement is another important aspect of installation of cable bolt. In pre-reinforcement, the cable bolt maintains the natural rock mass and increases the rock’s shear strength; while in post-reinforcement, the rock mass has lost most of its strength by unconstrained displacement at discontinuities (Windsor, 1992).

### 2.3 Types of cable bolts

Different types of cable bolt strand configurations have been designed to provide a support for different rock mass failure conditions. The plain strand cable bolt is the first type, which was developed many years ago but their poor performance was the reason to design other types of cable bolts (Hutchinson and Diederichs, 1996). The cable bolt design is based on selecting an applicable element, installation procedure and decision on using pre- or post- reinforcement in combination with pre- or post- tensioning. There are different types of cable bolts available in Australia as shown in Table 2-1. This table contains information about the shape, construction and strength properties of various cables used in mines and civil constructions.
Table 2-1 Different types of Cable bolts used in Australia (Jennmar, 2016, Megabolt, 2016 and Minova, 2015)

<table>
<thead>
<tr>
<th>Cable type</th>
<th>Longitudinal Section</th>
<th>Bulb spacing (mm)</th>
<th>Strand diameter (mm)</th>
<th>UTS Strand (t)</th>
<th>Drill hole diameter (mm)</th>
<th>Lay length (mm)</th>
<th>Elongation at strand failure (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indented TG bolt</td>
<td></td>
<td>-</td>
<td>28</td>
<td>60</td>
<td>42</td>
<td>400</td>
<td>5-7</td>
</tr>
<tr>
<td>Plain TG bolt</td>
<td></td>
<td>-</td>
<td>28</td>
<td>63</td>
<td>42</td>
<td>400</td>
<td>5-7</td>
</tr>
<tr>
<td>Plain SUMO</td>
<td></td>
<td>250</td>
<td>28</td>
<td>65</td>
<td>42</td>
<td>400</td>
<td>5-7</td>
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<td>Indented SUMO</td>
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<td>28</td>
<td>65</td>
<td>42</td>
<td>400</td>
<td>5-7</td>
</tr>
<tr>
<td>Plain Superstrand</td>
<td></td>
<td>-</td>
<td>21.8</td>
<td>60</td>
<td>42</td>
<td>300</td>
<td>6-7</td>
</tr>
<tr>
<td>Indented Superstrand</td>
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<td>60</td>
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## Table 2-1 continued: Different types of Cable bolts in Australia (Jennmar, 2016, Megabolt, 2016 and Minova, 2015)

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<th>Cable type</th>
<th>Longitudinal Section</th>
<th>Bulb spacing (mm)</th>
<th>Strand diameter (mm)</th>
<th>UTS Strand (t)</th>
<th>Drill hole diameter (mm)</th>
<th>Lay length (mm)</th>
<th>Elongation at strand failure (%)</th>
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<td><img src="image3" alt="MB7" /></td>
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<td>48</td>
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<td>-</td>
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<tr>
<td>Bowen Cable</td>
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<td>60</td>
<td>42</td>
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Table 2-1 continued: Different types of Cable bolts used in Australia (Jennmar, 2016, Megabolt, 2016 and Minova, 2015)

<table>
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<th>Cable type</th>
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<th>Strand diameter (mm)</th>
<th>UTS Strand (t)</th>
<th>Drill hole diameter (mm)</th>
<th>Lay length (mm)</th>
<th>Elongation at strand failure (%)</th>
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<td>Megastrand (Spiral, Bulbed)</td>
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<td>31</td>
<td>62</td>
<td>42</td>
<td>600</td>
<td>5-6</td>
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<tr>
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<td>62</td>
<td>42</td>
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<td>5-6</td>
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<td>52</td>
<td>600</td>
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</tbody>
</table>
2.4 Performance of cable bolts under axial and shear loads

The axial loading behaviour of cabled rock mass is complicated and it is controlled by the interface between the grout and cable bolt. There are additional factors such as face restraint anchors and internal anchors in the behaviour of cable bolts, which are shown in Figure 2-1. This figure shows how the rock mass displacement varies based on the interactions between all components of the system. The cable bolt has an inherent twisting action that becomes coupled with the torsional interaction between the outside surface of the peripheral wires and the grout. Therefore, the strand is pulled out during deformation due to the shearing of asperities and torsion by rotating the strand. When the axial load increases, the high axial load combined with torsional forces causes the shear through the peripheral wires and the grout. It is expected that the failure surface has a minor frictional response with mechanical interactions over an elemental length of the strand. The ‘debonding front’ is defined as the travel of shear and torsion mechanism from the point of load application. The strength of the grout controls the propagation of debonding. Friction and radial stress control the behaviour of the failure surface behind the propagating front. The steel surface roughness and the grout particle size affect the friction. Increasing the grout particle size provides a higher coefficient of friction however it decreases grout strength, reduces the strength of the interface and modifies the radial stress created during dilation. During cable bolt installation and grout curing, a radial stress is set up which depends on different factors during loading such as: composite Poisson’s ratio of the strand, torsional rigidity of the cable, compaction and dilation of failed grout at the interface, and changes in the rock mass stress over time (Windsor, 1992).
Hutchinson and Diederichs (1996) discussed three different types of loading subjected to the cable bolt as; axial or tensile, shear and a combination of the axial and shear loads. The behaviour of cable bolt can be analysed individually or in combination with an array of cable bolts.

2.5 Experimental studies on the axial performance of cable bolts

Axial loading tests of cable bolts can be conducted in the field or in the laboratory using two different methods. The first method is to grout one end of cable bolt in the pipe and keep the other end free for pulling test. The free end can easily rotate (Maloney et al., 1992). Second method is a non-rotating and double pipe (Villaescusa et al., 1992). This test is more complex compared with the first type. The first method gives the lower bound strength because the free end of cable can rotate freely. The load transfer capacity of non-rotating system is higher compared with the rotating method, as shown in Figure 2-2, because the rotating method underestimates the load transfer of cable bolts however the non-rotating method overestimates the load transfer of cable bolts.
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Literature review of performance of cable bolts- Experimental studies

Figure 2-2 Comparison between rotating and non-rotating pull testing results (Bawden et al., 1992)

Over the years, various methodologies have been developed to study the load transfer capacity of cable bolts with different parameters, comprehensively. Following factors control axial pull-out tests on the fully grouted cable bolts:

- Type of cable bolt,
- Grout properties, such as water to cement ratio, curing time and UCS,
- Length of embedment (anchor length or free pull length),
- Material property, borehole diameter and field test properties,
- Rate of pulling,
- Load and displacement during pull out test, and
- Responses of cable bolt, such as rotation, stick slip and cable strand rupture.

The schematic pull out failure modes of the cable bolt in is shown in Figure 2-3. There are four modes of failure:
- Failure at cable to grout interface,
- Failure through grout column,
- Failure at grout to rock interface, and
- Failure through rock around borehole wall.

![Four failure modes of load transfer in cable bolts (Thomas, 2012)](image)

**Figure 2-3** Four failure modes of load transfer in cable bolts (Thomas, 2012)

The failure in the cable-grout interface is the main reason and the most common failure type of cable bolts in the field (Hyett *et al.*, 1996; Singh *et al.*, 2001). This is due to the insufficient frictional resistance between the cable and the grout and also the lack of quality control at the installation time (Rajaie 1990, Hyett *et al.*, 1995, Hyett *et al.*, 1996, Hutchinson and Diederichs 1996, Singh *et al.*, 2001 and Chen *et al.*, 2016a, 2016b).

Thompson and Windsor (1995) studied the effect of pretension load on the load transfer of cable bolts subjected to axial loading. The results demonstrated that pretension load does not affect the internal stiffness of the cable bolt because this factor is dependent on
the load transfer features between the cable bolt and the rock. On the other hand, pretensioning has the following advantages:

- Develop flexibility to blast vibrations by plate holding the surface restraint system rigidly against the rock surface,

- Reduces discontinuous opening and increase the shear strength of rock mass and the overall strength of the system.

Mirabile et al. (2010) achieved the same result as Thompson and Windsor (1995). Cable bolts with pretension loads (0, 17.8, 35.6, 53.3, 71.1 and 88.9 kN) were tested using barrel and wedge on each end of cable bolts. The effect of pretension load on the total deflection at failure of cable bolt and the constant stiffness of the cable bolt material were investigated using a Tinius Olsen testing machine. The pretension load was applied when the cable bolt material is in the elastic stage and therefore, it did not affect the overall load transfer of cable bolts as shown in Figure 2-4.

![Figure 2-4 Load versus displacement curves for a cable bolt subjected to different pretension loads (Mirabile et al., 2010)](image_url)
Generally, pull testing has been adopted in two methodologies of constant normal stiffness (CNS) and constant normal load (CNL). In the first method, the cable bolt is embedded in steel or in a rigid tube or pipe. In the second method, a pressurised biaxial cell is used (Moosavi, 1997 and Holden and Hagan, 2014).

The following parts include the review of previous studies on unconstrained single embedment and constrained embedment pull out testing.

### 2.5.1 Unconstrained single embedment pull tests

The unconstrained single embedment pull test is the simplest method of testing cable bolts in rock anchor, which allow the cable to rotate relative to the grout. This method consists of two parts; the first part is the embedment cable bolt in a confining medium, such as rock or metal tube and the second part is the free rotating part of cable bolt to apply a pulling load (Hagan and Chen, 2015).

Stillborg (1984) conducted short and long single embedment length pull-out tests of cable bolts using concrete blocks to provide the confinement instead of rock mass. The advantage of using concrete block by Stillborg instead of steel tube by Fuller and Cox (1975) is that concrete simulates rock mass properties better than the steel tube. Another benefit of this model was that the efficiency of concrete in simulating the borehole roughness and radial stiffness. It was concluded that the surface profile of cable bolts, grout type and curing conditions affect the load transfer performance of cable bolts although the embedment length does not affect it directly. The free end of cable bolts allowed rotation and separation of cable bolts under load which is the disadvantage of this method.

Farah and Aref (1986) continued with the methodology proposed by Stillborg (1984). Their aim was to simulate the dynamic loading environment by using a fast loading rate.
They studied the effect of rock mass on the ultimate bonding strength. Therefore, a mix of sand, water and cement as a grout mix versus a mix of sand, water, cement and aggregates as a concrete mix were used. It was concluded that the mix of concrete provides higher ultimate bonding strength, ductility and failure load in a dynamic loading environment compared with the mix of sand, cement and water.

Hassani and Rajaie (1990) studied the effect of using shotcrete instead of aggregates on the load transfer capacity of cable bolts. The result of this study indicated higher residual bond strength for the grouts using shotcrete instead of aggregate, although the bond stiffness and bearing capacity decreased.

Rajaie (1990) studied the relationship between the anchorage strength of cable bolt and the diameter of the rock sample around the cable bolt. Plain cable bolts were used for this study and it was found that the load transfer capacity of cable bolt was nearly constant when the diameter of the rock sample was more than 250 mm. The drawback of this study was that the effect of rock sample size on the modified cable bolts such as bulbed or birdcage bolts were not studied.

Benmokrane et al. (1992) used six different types of grout and two kinds of reinforcing tendons including seven-wire cable and a deformed bar to study the effect of grout on the bond stress-slip relationship. The rock mass was simulated using a 200 mm diameter concrete cylinder (Figure 2-5). Results showed variations between the load transfer for cable bolts and steel bars.
Figure 2-5 Reinforced concrete cylinder with cable bolt and steel bar (Model A is a cable bolt and model B is a steel bar) (Benmokrane et al., 1992)

Hyett and Bawden (1996) conducted 75 tests to study the impact of bulb frequency on the ultimate pull test capacity of Garford cable bolts and to compare these results with plain strand cable bolt results. The embedment length of cable bolts was 600 mm, 900 mm and 1800 mm, respectively. The grouted cable bolts were allowed to cure for 26 days. The grout with the ratio of 2:5 for water to cement was constrained by the use of Schedule 80 steel pipes and Schedule 40 aluminium pipes for the constant radial stiffness boundary conditions purpose. It was observed that bond stiffness was independent of test parameters when the loads were less than 60 kN. Although the rate of loading during the cable pull test increased for higher amount of loads. The stiffness of the grouted cable bolt was higher when the bulb spacings were closer, length of embedment was longer and radial stiffness of the confining medium was higher. For samples with two or more bulbs along the embedment length no slip was detected.

Moosavi (1997) and Moosavi et al. (2001) conducted a series of pull-out tests on deformed bars and modified cable bolts to study the bond failure mechanism of grouted
rebars and cable bolts. The result revealed that the leading parameter in monitoring the behaviour of rebars and cable bolts was the cement shearing. Thus, it was concluded that studying the shear behaviour and mechanical properties of Portland cement are essential. Less research had been conducted on the properties of grout without any aggregates although various studies had been conducted on the mechanical properties of concrete. It should be stated that the addition of aggregates may influence the grout steel bonding effectiveness as the aggregate surfaces could be slippery.

Ito et al. (2001) continued research by Benmokrane et al. (1992) in more details by testing four different types of bolt (deformed bolt, twist bolt, plain strand cable bolt and bulb strand cable bolt) using artificial rock (Figure 2-6). The X-Ray medical imaging technique was used to reconstruct the image of the cross-section of the object. It was observed that the mode of failure in the cement grout was dependent on the type of bolt. The failure in the grout was spiral with the twist for deformed and twist bolts. The failure in the cement grout for plain strand and bulb strand cable bolts was the radial splitting mechanism.
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Figure 2-6 Schematic of the modified LSEPT pull test (Ito et al., 2001)

Hyett and Bawden (1996) studied the effect of bulb spacing on the performance of Garford bulb cables. The result showed that the axial load and bond stiffness increase when the bulb spacing is reduced, the embedment length is longer and the radial stiffness is higher (Figure 2-7).
Kent and Bigby (2001) studied the standard birdcaged cable bolts installed in cementitious grout in rock subjected to pull-out tests by using weaker coal samples. The steel split tubes were replaced with the biaxial cell and also the embedment length was increased for appropriate grout tendons testing. The instrument was adopted in a way to conduct tests on the cable bolt with varied pretension loads. It was indicated that pretensioning under certain situations in UK coal mines improves the performance of system significantly. A key for successful reinforcement design is good *in situ* bond strength. The limitation of this study was that no bond failure was observed up to the yield point and no information was provided with regard to the nature of the failure mode. The project time was limited and developing the instrumentation for measuring load in cable bolts were time consuming. The result was also unacceptable for few cases where they were installed.
Prasad (1998) studied the influence of stress change on axial performance of plain and bulbed cables by using soft and hard rocks. The effect of stress change on plain cable bolts in the weaker rock was higher compared with the stronger rock; however this effect was negligible for the bulbed Garford cable.

Aoki et al. (2002) conducted long embedment pull tests on the standard plain strand and 0.5 m spaced bulbed strand cable bolts to investigate the variations between the non-linearity in in situ pull test and linearity in short embedment length pull tests. The grout with water to cement ratio (W:C) of 2:5 and the block with dimension of 4 m x 1.2 m x 0.6 m were used for this study as shown in Figure 2-8. It was found that the pull-out force of bulbed cable was 35% higher than the plain cable bolt in a competent rock, however no difference was observed in the weak rock conditions due to low confining stiffness.

![Figure 2-8](image.png)

**Figure 2-8** Long embedment pull test (Aoki et al., 2003)

Chen and Mitri (2005) studied the load transfer capacity of the seven-standard cable bolt and determined the effect of embedment length, W: C ratio and borehole diameter. The cement grout with embedment length of 152 mm and polyester resin with
embedment length of 185 mm were used to compare their performance. This difference in the embedment length and the limited number of tests with resin based grout were the main reasons for uncertainty in the result of this study.

Mosse-Robinson and Sharrock (2010) studied the effect of large borehole diameters on various cable bulb densities and poor quality grout conditions. An Avery pull testing machine was used to pull test 37 Garford cable bolts grouted into steel pipes to a depth of 1000 mm. The failure in the conducted tests occurred at the cable-grout interface and the rupture of cable strands. It was concluded that the size of the borehole does not affect the load capacity of a cable bolt which was also concluded by Rajaie (1990) by testing plain strand cable bolts in boreholes with size of 20 mm and 60 mm.

Tadolini et al. (2012) studied the impact of indentation geometry on the load transfer capacity of cable bolts. It was observed that the indentation depth increases the bearing capacity and stiffness of the cable bolt due to increasing the mechanical interlock at the surface between the cable bolt and grout as shown in Figure 2-9.

![Anchorage capacity testing results of standard versus indented PC-Strand](Tadolini et al., 2012)

Figure 2-9 Anchorage capacity testing results of standard versus indented PC-Strand (Tadolini et al., 2012)
Holden and Hagan (2014) followed up the study by Rajaie (1990) by conducting 16 tests on artificial rock cylinders with the length of 300 mm and diameters of 150, 215, 300 and 450 mm (Figure 2-10). Two modes of failure of grout/rock and cable/grout interfaces occurred. The majority of failures occurred at the grout and rock surface and only three failures occurred in the cable/grout interface.

![Cable bolt installation and cross section of resin column and cable bolt](image)

**Figure 2-10** Cable bolt installation and cross section of resin column and cable bolt

(Holden and Hagan, 2014)

Although Lardner and Littlejohn (1985); ISRM; suggested rotating pull testing of cable bolts is not a suitable methodology for investigating the load capacity of cable bolts because of rotation in the free ends. Thus, the rotationally constrained tests are better methodologies for testing the load transfer capacity of cable bolts. When the encapsulated length of a cable bolt is less than seven times the cable bolt diameter, the cable bolt will unwind from the confining medium which provides lower maximum pull-out load as shown in Figure 2-11 (Stillborg 1984).
2.5.2 Rotationally constrained tests

Four different methodologies are available for rotationally constrained pull tests, including:

- Split-Pipe pull/push tests,
- Modified split-pipe pull/push tests using a Hoek cell,
- Double embedment pull tests, and
- Short encapsulations pull tests.

This part of the study consists of the review of the studies conducted using these methodologies.

2.5.2.1 Split-Pipe pull/push tests

Fuller and Cox (1975) designed the earliest “split-pull” testing to study the load transfer performance of cable bolts. The confinement of the grouting materials around the cable
A bolt was provided by the steel split pipes. This system was able to constrain the rotation of the cable bolt but the amount of confining pressure provided by the steel tube was much higher than the rock mass in *in-situ* conditions (Figure 2-12). Thus, the peak load measured by this method was higher than the real value achieved in the field. The shape and conditions of the cable bolt were introduced as the critical factors on the load transfer performance of cable bolts.

Additionally, Cox and Fuller (1977) studied the effect of wire indentations on the mechanism of load transfer in cable bolts. The indentation was introduced as an important and useful factor in the load transfer performance of mill-finished wires. This was different for the rusted wires because the indentation reduced the effective rusted surface area. Therefore, a slightly rusted indented cable bolt was suggested. Another
result of this research was the positive impact of high grout strength on the load transfer capacity.

Goris (1990) studied the axial performance of cable bolts under constant radial stiffness boundary conditions to prevent rotational failure in the cable/grout interface. The following results were obtained:

- Bearing capacity increased linearly by increasing the embedment length from 203.2 to 812.8 mm.
- Bearing capacity increased with the presence of double cables, high curing temperatures, low water-cement ratios and sand-cement grouts instead of cement grout.
- The breather tube size filled with grout did not affect the load transfer capacity of cable bolts.

Reichert (1991) designed the “split-push” test as a modification of “split-pull” test developed by Fuller and Cox (1975). In this method, instead of pulling, the grout column and pipe were pushed off from the cable (Figure 2-13). The result demonstrated that the higher radial confinement provides greater load transfer capacity. The stiffer grout with the water-cement ratios of less than 1:2.5 increased the cable bolt’s load transfer capacity from 50% to 75%.
Figure 2-13 Split-push test apparatus (Reichert, 1991)

The credibility of the method proposed by Reichert (1991) was challenged by Bawden et al. (1992). It was observed that the load capacity of cable bolts in short embedment lengths in situ tests at the Golden Giant Mine in Canada was lower in comparison with laboratory results using the Schedule 80 steel split-pipe test. This inaccuracy was the reason for developing Schedule 40 aluminium and PVC pipes with lower radial stiffness (Figure 2-14) compared with Schedule 80 steel to represent the in situ conditions. It should be recognised that the push testing method is not considered as a realistic method of testing tendons as it does not reflected the reality of tensioning in situ, and that the load transfer capacity evaluation is generally different from pull testing as reported by Aziz et al. (2005).
Hyett et al. (1993) used the “split-push” test equipment developed by Reichert (1991) to test various types of modified cable bolts. It was found that the impact of high water to cement ratios and lower confinement was less on the nutcaged cable bolts. Increasing the size of nutcase increases the stiffness of nutcase cable bolts. Hyett and Bawden (1994) continued their study on 25 mm Garford bulb cable bolts to investigate the effect of low radial confinement and low water to cement ratio. The result demonstrated a little dependency of the load transfer performance of Garford bulb cable to the radial confining pressure and high water to cement ratio of 1:2.
The behaviour of cable bolts in various studies conducted by Fuller and Cox (1975), Goris (1990), Hyett et al. (1992), Satola (2007) and Faulkner et al. (2013) was stiffer than the field condition, which is not a desirable variation. This is due to confining both ends of the cable bolt to prevent rotation during pull-out tests. The difference between this test and the *in situ* test is that the stress-strain behaviour of pipe used for this test has different behaviour than the performance of rock in the field (Goris 1990).

*2.5.2.2 Modified split-pipe pull/push tests using a Hoek cell*

Macsporran (1993) developed a Modified Hoek Cell (MHC) to study the performance of cable bolts under constant normal load conditions. This improvement was the result of using metal pipes, concrete blocks or rock mass to provide the confinement to the cable as shown in Figure 2-15. It was observed that the load transfer capacity of cable bolts was increased by increasing the confining pressure and decreasing the water: cement ratio.

*Figure 2-15* Modified Hoek Cell (Macsporran, 1993)
2.5.2.3 Double embedment pull test

Hutchins et al. (1990) proposed double embedment pull testing to examine the load transfer capacity of the birdcaged cable bolt as shown in Figure 2-16. This method was different from other pull testing methodologies, because it allowed the study of the impact of embedment lengths on both sides of the discontinuity however this method had limitation in simulating underground conditions, especially for the grout/rock interface. This is because the system was threaded internally and there was no failure in the interface. Unless the two lengths are embedded at different length, the validity of the test will be questioned and the load reading on the testing machine will not be valid.

![Figure 2-16 Double embedment length set up by Hutchins et al. (1990)](image)

Renwick (1992) compared the performance of an Ultrastrand cable bolt with plain and birdcage cable bolts, using the same methodology as suggested by Hutchins et al. (1990). As shown in Figure 2-17, the embedment length was two metres and the load was applied to both ends. The result from this study was not sufficient because the
embedment length of the plain and birdcage cable bolts were different from the Ultrastrand cable, which made the comparison complicated.

![Figure 2-17 Dual loaded double embedment pull test (Renwick, 1992)](image)

Martin et al. (2000) conducted laboratory tests on three double embedment pull tests on instrumented king wire cables encapsulated in concrete blocks using resin grout by mounting strain gauges at 0.15, 0.35, 0.61, 0.91 and 1.22 m from the head of the bolt on the king wire (as shown in Figure 2-18) of the cable to study the distribution of load along the cable bolt. The use of the instrumented cable bolt was successful and load changes and deformation of the adjacent rock mass were depended to each other. The limitation of test numbers was the main reason of unreliability of this research.
Figure 2-18 Calculated load and strain gauges on the king wire of the 15.2 mm cable bolt (Martin et al., 2000)

Aziz et al. (2016a) studied the load transfer characteristics of plain and spiral cable bolts. The aim of this study was to introduce a new non-rotating pull testing apparatus because all the previous methods consist of the wastage of material; firstly, the wastage due to steel tubes because they can be used once only and secondly for the amount of wastage of rock material or composite materials such as concrete blocks. The new proposed method is a non-wasting materials method, which requires grout/resin for cable bolt installation (Figure 2-19). Figure 2-20 and Table 2-2 show the results of the pull testing of plain and spiral Superstrand cable bolts four days, one week and one month after the cable bolt’s encapsulation. The spiral cable bolt had higher peak load compared with the plain one, however the displacement at peak load was much lower. Moreover, irrespective of the cable type, the peak load increased by increasing the number of encapsulation curing days.
Figure 2-19 New non-rotating pull testing apparatus (Aziz et al., 2016a)

Figure 2-20 New non-rotating pull testing results (Aziz et al., 2016a)
Table 2-2 New nonrotating pull testing results (Aziz et al., 2016a)

<table>
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<th>One week encapsulation</th>
<th>One month encapsulation</th>
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<td>Spiral</td>
<td>Plain</td>
</tr>
<tr>
<td>Peak Load (kN)</td>
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<td>150.8</td>
</tr>
<tr>
<td>Displacement (mm)</td>
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<td>2.5</td>
<td>46.0</td>
</tr>
<tr>
<td>Bond strength (kN/mm)</td>
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<td>0.87</td>
<td>0.887</td>
</tr>
</tbody>
</table>

2.5.2.4 Short encapsulation pull tests

The main issue with pull testing of a cable bolt is the free or un-grouted section of the cable bolt, which causes rotation due to its low torsional stiffness. Therefore, the bearing capacity in this method is lower than the real value of the non-rotating one. Clifford et al. (2001) developed a Laboratory Short Encapsulation Pull Test (LSEPT) to overcome this issue and to determine the load transfer performance of fully grouted cable bolts and to simulate the performance of cable bolts for in situ rock conditions. This system consists of two sections of embedment and anchor sections (see Figure 2-21). This method has been used by various researchers in the United Kingdom, France, and Australia by Altounyan and Clifford (2001), Bigby (2004), Bigby and Reynolds (2005), Blanco Martin et al. (2012), Martin (2012) and Thomas (2012). The benefit of the LSEPT method is that the end of the cable is embedment in a material cylinder instead of other methods where the cable is embedment in a rigid or semi-rigid steel tube. This method does not constrain any lateral dilation (Holden and Hagan, 2014). The stress distribution along the rock mass in the field occurs due to the lateral dilation stress activated by axial loading of the cable bolt.
Martin et al. (1996) used resin-grout to study the load transfer capacity of Garford cable bolts. Concrete blocks with UCS of 35 MPa were used as artificial rock.

The following results were concluded:

- The stiffness of the cable bolts with the button indentation was higher.
- The borehole diameter affected the load transfer capacity of cable bolts. The load transfer capacity was nearly constant when the borehole diameter was between 25.4 and 35 mm. The load transfer capacity decreased when the borehole diameter increased from 42 to 106 mm. Mosse-Robinson and Sharrock

**Figure 2-21** Laboratory Short Encapsulation Pull Test setup (Clifford et al., 2001)
(2010) found similar result as the load transfer capacity increases when the borehole diameter is smaller.

Ito et al. (2001) subjected axial loading on the bulb and plain standard cable bolts at a rate of 0.05 kN/min until failure using a hollow-ram jack with an anti-rotational device (Figure 2-22). Cement paste block as an artificial rock was used to represent *in situ* rock mass confinement conditions.

**Figure 2-22** Modified LSEPT pull test (Ito et al., 2001)

Bigby and Reynolds (2005) reported that double embedment was “very artificial” with unsatisfactory results in *in situ* conditions due to the omission of the interaction between the borehole wall and grout compared with the LSEPT.
Thomas (2012) studied the load transfer characteristics of 14 types of post-groutable cable bolts in use in the Australian coal mining industry. Various studies on this topic such as Double Embedment Pull Test (DEPT) and LSEPT were reviewed in this study (Figure 2-21 and 2-23). The DEPT method was not suitable to study the load transfer of cable bolts because it is not capable of assessing the grout and rock interface. The limitation of the LSEPT method was that the free end of the cable bolt could rotate and it was possible to unwind the cable bolt out of the core under load specifically plain-strand cable. Moreover, the confining applied by the biaxial pressure cell was fixed at 10 MPa which is not necessarily a true value and does not replicate the ground conditions. The confining pressure is a dynamic value in the cable bolt life however it was constant in this study.

![Double Embedment Pull Test](image)

Figure 2-23 Double Embedment Pull Test setup (Thomas, 2012)
Thomas (2012) modified the available method and proposed another method as shown in Figure 2-24. Sandstone with diameter of 142 mm and a UCS value of between 19 and 25 MPa was used. 450 mm of the steel was grouted in the ram casing and 320 mm of embedment length in the sandstone. The barrel and wedge was used to fix the cable. Rifling was applied to the system with six different drill bit sizes. Cable Bolt Grout (CBG) with strength of 80 to 100 MPa was used. The hydraulic ram applied load to the sample before yielding of the anchor at an approximate rate of 10 kN per second. The results showed higher capacity of up to 400% for bulbed or nutcaged cable bolts compared with the plain strand cables. It was observed that increasing the size of the borehole increased the capacity of the system because the strength of the grout is mostly higher than the rock mass strength.

**Figure 2- 24** Modified version of the Laboratory Short Encapsulation Pull Test incorporating the steel cylinder (Thomas, 2012)
Chapter II  Literature review of performance of cable bolts- Experimental studies

Hagan et al. (2014) reviewed the pull testing research conducted by previous researchers and concluded that the rock mass confinement, cable surface geometry, water: cement ratio and embedment length have been studied however the effect of different available types of cable bolts had not been studied, precisely.

Craig and Holden (2014) studied the bond strength of cable bolts marketed in Australia in situ at Baal Bone mine. A non-rotating system similar to the method proposed by Thomas (2012) was used for in situ cable bolt pull-out testing. Indented 22 mm Superstrand, indented TG cable 28 mm hollow strand, indented SUMO 28 mm hollow strand with a 35 mm diameter nutcage and plain wire SUMO 28 mm hollow strand with a 35 mm diameter nutcage were used for this study. The indented SUMO cable bolt had higher bond performance compared with indented Superstrand and indented TG cable bolts. Indentation of cable bolt increases its stiffness by 50% and more in comparison to plain wire cable bolts.

Craig and Murnane (2013) used 400 mm embedment length in a weak coal roof. 400 mm embedment length was used for the in situ condition, which is mostly in moderate strength mudstone. The resin with a UCS of 70 MPa and cementitious grout with strength of 30, 55 and 70 MPa at curing days 1, 7 and 28 were used. An anti-twist SEPT in situ method was developed as shown in Figure 2-25. Four different types of cable bolts were used for this study, including indented Superstrand, indented TG, indented SUMO and plain SUMO cable bolts. A 20% increase in the load transfer capacity of cable bolt by this new developed method was reported. The strength of SUMO cable bolt was higher than indented Superstrand and indented TG cable bolts.
Chen et al. (2016a) modified the pull testing methodology developed by Thomas (2012) and developed a new LSEPT to understand the load transfer performance of fully grouted cable bolts over a large pull-out displacement range of 100 mm. This new system was capable of determining the peak and residual capacity of a fully grouted cable bolt. Different types of cable bolts such as a modified (MW9) and a plain Superstrand cable bolt were tested including the slippage along the interface of cable/grout and also grout/rock. The weak rock-mass with a low strength of 8 MPa was tested which could be a possible case in coal mines. In this method, concrete was used instead of sandstone (see Figure 2-26) because it was possible to cast a group of

---

**Figure 2-25** Developed anti-twist method (Craig and Holden, 2014)
samples at the same time to assure that all the samples have the same conditions, which were not possible when using sandstone.

The diameter of the cylinder sample was changed from 142 mm to 300 mm because the size of the sample can affect the load transfer of the cable bolts. Samples with various sizes ranging from 150 mm to 500 mm were tested using indented SUMO cable bolts manufactured by Jennmar Australia. Results show that when the sample size was 300 mm in diameter or over, the sample size did not affect the load transfer capacity of the system. Accordingly, the sample size was changed to 300 mm in the new method (Urrahman et al., 2015). It was observed that the failure occurs in the grout/rock interface when the rock is weak while the failure occurs in the grout/cable interface for the strong rock. Cable type is an important factor in the performance of the system and the peak capacity was higher for the bulbed cable compared with the plain cable bolt.

![Figure 2-26 Front and side views of the new LSEPT facility (Chen et al., 2016a)](image-url)
Chen et al. (2016b) continued the study by Chen et al. (2016a) and used the same methodology for their tests. The MW9 cable bolt with Stratabinder HS grout with a UCS of 54 MPa was used to conduct LSEPT. All failures occurred at the grout/cable interface. These are the following conclusions:

- The initial stiffness of the system was not dependent on the embedment length; however, the peak load was. The peak load increased when the embedment length increased.
- Initial stiffness and the averaged peak load for strong and weak surrounding material was compared. It was observed that both parameters were higher for a strong rock mass.
- Increasing the size of the borehole in a weak material increased the axial performance of the MW9 cable bolt. For the normal borehole size, the failure occurred in the grout/rock interface however for the large borehole, it occurred in the cable/grout.

2.6 Experimental studies on the shear performance of cable bolts

Shear testing is a more complex method if compared with the axial pull tests, because it requires a special facility of restraining frame and hydraulic or screw feed actuators. There are many important factors of cable bolt behaviour subject to shear in the sense of shear and dilation displacements including the orientation of the cable bolt in regard to the test interface and motion direction (Hutchinson and Diederichs, 1996). There are several important factors such as lay length, Ultimate Tensile Strength (UTS), cable bolt profile surface, grout or resin type, pretension load, cable angle to the joint, rock mass strength and borehole quality in regard to shear strength of cable bolts.
Windsor (1992) conducted a series of tests to determine the effect of angle between the plain strand and the shear plane. This angle affects the efficiency of cable bolts directly. Results from these tests are shown in Figure 2-27.

![Figure 2-27 Results from shear tests versus bolt orientation (Windsor, 1992)](image)

Fuller and Cox (1978) measured the tension in the cable bolt oriented at various angles to the shearing discontinuity. Bjurstrom (1974) reported that when the angle between the bolt and shear plane is less than 35° degrees, the bolt failure occurs in tension. In addition, Fuller (1983) stated that the angle between the shear plane and cable bolt should be between 15° and 30° degrees to achieve higher strength (Figure 2-28).

![Figure 2-28 Effect of shear resistance on cable inclination (Fuller, 1983)](image)
Stillborg (1984) conducted a series of single shear tests to determine the shear behaviour of fully grouted cable bolts. The granite blocks with Young’s modulus of 68 GPa and a UCS of 234 MPa were used to encapsulate the tested bolts. The cable angels with respect to the sheared joint surfaces were 45° and 90° degrees (see Figure 2-29).

![Diagram of loading geometry and angle θ in the shear tests (Stillborg, 1986)](image)

**Figure 2-29** Loading geometry and angle θ, in the shear tests (Stillborg, 1986)

Larger size blocks were cut in two halves and perfectly matched with smooth surfaces and cross-sectional area of 0.6 m². The normal load was kept constant at 200 kN and the shear load was increased up to 100 kN and then it remained constant as shown in Figure 2-30.

It was determined that when the angle of cable to the joint surface was 45° compared with 90°, the shear resistance of the grouted cable bolt was significantly higher. Also, the maximum of shear resistance in the cable occurred after 10 mm of shear displacement.

Four different modes of failure for a fully grouted cable bolt were introduced as:

- Rock mass,
- Rock/grout bond,
- Grout/cable bond, and
- Cable bolt.

The first two failure modes are not common while the third failure model (cable interface bond) is the most common.

![Shear and normal load as a function of shear displacements (Stillborg, 1984)](image)

**Figure 2-30** Shear and normal load as a function of shear displacements (Stillborg, 1984)

Goris *et al.* (1996) conducted a series of direct shear tests on 15.24 mm diameter cable bolts with 25.9 mm hole diameter using concrete blocks of 69 MPa strength and the joint surface area of 0.078 m² as shown in Figure 2-31. The yield strength of cable bolt was 258 kN.
It was observed that yield occurred in 4 mm displacement at 220 kN. The result from the single shear test was higher than the double shear test for the same type of cable bolt. This occurred due to higher shear resistance of single shear test. As shown in Figure 2-31, zone “A” shows the point of concentration of load in the joint. The cable bolt did not fail during the test because the maximum shear displacement was set at 46 mm and the peak shear load was not measured.

Craig and Aziz (2010) conducted a series of double shear tests on 28 mm TG hollow strand cable bolts. Cable bolts are under tensile and shear load in mines because of the roof deformation loads; therefore, two tests were conducted to investigate the shear behaviour of cable bolts with different displacement limitation and initial pretension load. The Jennmar TG cable developed in 2007 is a post grouted bolt with 9 wire
strands (each element is 7 mm in diameter) rated at 618 kN. The two metre long tested cable bolt section was 28 mm in diameter. Three concrete blocks 50 MPa in strength were cast and tested. The dimensions of corner blocks were 300 x 300 x 300 mm³ while it was 450 x 300 x 300 mm³ for the middle block. Conbextra CB “hi-thix” grout was used for both tests. The initial pretension load of these cable bolts were 50 kN and 90 kN respectively. The maximum shear displacement was 50 mm and 75 mm, respectively.

Figure 2-32 shows the result of the test with 50 kN initial pretension of axial load and maximum displacement of 50 mm and 90 kN initial pretension axial load with 75 mm maximum displacement.
Figure 2-32 Results of double shearing test of 28 mm hollow cable bolts a) initial normal load of 50 kN and maximum displacement of 50 mm, b) initial normal load of 90 KN and maximum displacement of 75 mm (Craig and Aziz, 2010)

The maximum displacement for the first test was 50 mm with 900 kN and 238 kN for the shear and axial loads respectively. The failure in the second test was at 60 mm displacement and 1354 kN and 385 kN of shear and axial loads respectively. It was observed that all failures occurred in tension and there was no failure in shear. Only the failure in the central core was in shear. The concrete block was also crushed about 60 mm inward as shown in Figure 2-33. On the other hand, Fuller (1983) stated that pre-
tensioning of the cable bolt is a time-consuming and difficult process that only provides support to rock mass, which is close to the already stabilised condition *in situ* and therefore, it is only redundant.

![Concrete/grout crushing and cable bolt deformation in the vicinity](image)

**Figure 2-33** Concrete/grout crushing and cable bolt deformation in the vicinity (Craig and Aziz, 2010)

Aziz, *et al.* (2014) compared the performance of Hilti 19 wire HTT-UXG plain Superstrand and Hilti 19 wire HTT-IXG spiral indented Superstrand cable bolts with 22 mm diameter and 60 t of tensile strength by conducting double shear test. Minova/Orica FB400 pumpable grout was used to encapsulate cable bolts in 40 MPa concrete blocks. The shear displacement was limited to 70 mm with 50 kN initial axial load. The procedure for testing was exactly the same as reported previously by Craig and Aziz (2010).

The maximum shear and axial loads for indented cable bolts were 904 kN and 254 kN at 52 mm of shear displacement. The failure occurred at a combination of shear and
tension. This failure occurred, because the cable was bent in the vicinity of the surface where the concrete was crushed. On the other hand, the maximum of shear and axial load for the plain cable bolts were 1024 kN and 400 kN at 75 mm shear displacement. The indented cable bolt had lower values in shear and tension. Also, the result of the tensile test of the single strand shows that the strength of indented strand was 10% less than the plain one. No rotation was detected in any of tests.

Aziz et al. (2015a) investigated the performance of 19 wire 21.8 mm diameter plain and spirally profiled Hilti cable bolts subjected to double shear tests. Figure 2-34 shows the result of the plain Hilti cable bolt. The result for the plain cable bolt was higher than the spiral cable bolt because of the reduction in the cross-sectional area of spiral cable bolt and also the tensile strength of plain cable bolts was higher than the spiral cable bolts. The result of double shear tests was also compared with single shear tests by British Standard (BS 7661-2- BSI 2009). Figure 2-35 shows this comparison. It was found that the single shear tests underestimate the shear strength of Superstrand cable bolts. Dash lines show the result of single shear tests and solid lines show double shear test results.
Figure 2-34 Shear and axial loads versus shear displacement for plain Hilti cable bolt in double shear test (Aziz et al., 2015a)

Figure 2-35 Comparison between single and double shear tests. Dotted lines graphs are from BS single shear apparatus results (Aziz et al., 2015a)
2.7 Summary

The performance of cable bolts in tension, shear and their combination was studied in this chapter. Various studies were mainly based on tensile performance of cable bolts by pull testing. Experimental studies were conducted to determine the performance of cable bolts in tension; however studies on shear performance of cable bolts were limited.

The main findings from the review of performance of cable bolts are summarised as:

- The load transfer capacity of cable bolts depends mainly on the shear resistance at the interface between the cable bolt and the grout. There are three different important factors on the load transfer capacity:
  - Resin/ cementitious grout adhesion
  - Mechanical interlock
  - Friction (Gambarova, 1981)

The first factor is the least important one because by just a very small displacement of 0.2 mm, the influence of chemical adhesion will change (Fuller and Cox, 1975). The second and third factors have more influence (Stillborg, 1984). The mechanical interlock increases when the movement within the borehole between the cable bolt and cement grout increases and an extra normal load being generated in the interface of cable and grout. Friction occurs in the interface between the cable and the grout, which prevents the cable bolt from slippage and therefore the load transfer capacity of modified cable bolts with spiral or indented profiling will be higher than the plain cable bolt.

- The main important factors in the failure of cable bolts in shear are in grout/cable bond, and cable bolt although the failure in the cable bolt is the most common one. The shear strength of cable bolts is related to the cable surface profile type,
number of wires, ultimate tensile strength, the borehole quality, rock mass strength and grout properties. It was observed that the plain cable bolt has higher shear strength than the indented one.

- Some conflicts were observed in various research results such as the effect of pretension load on peak shear strength of cable bolts. This was the reason for obtaining additional information, especially in mathematical simulation. Thus, Chapter III focuses on the use of analytical simulations on the performance of tendons.

Thus, this thesis focuses on the experimental studies on determination of the shear performance of various types of cable bolts with different pretension loads subjected to single and double shear tests. Analytical and numerical simulations are proposed to simulate the performance of cable bolts in shearing at different stages of loading.
Chapter III

LITERATURE REVIEW OF THE MATHEMATICAL
SIMULATIONS OF TENDONS

3.1 Introduction

This review presents the mathematical simulations of tendons to include solid rock bolts and cable bolt strands and their behaviour under axial and shear loading conditions. The mathematical simulations on rock bolts involve limited analytical simulations being conducted on cable bolts, bearing in mind that most of the current simulations are made on solid rebar (rock bolts). These simulations have been adopted for cable bolts taking into consideration only one wire of cable strand can be considered as the solid rebar of smaller diameter particularly on shearing.

3.2 Mathematical simulations on axial performance of rock bolts by pull-out tests

Hawkes and Evans (1951) conducted a series of pull out test to study the behaviour of a fully grouted resin bolt and the anchorage bond in reinforced concrete column. An exponential function was introduced for the bond stress by using pull-out tests. The results showed that the maximum of the bond stress occurred at the free end before any slippage occurred. The equation is:

\[ \sigma = \sigma_0 \exp\left(-\frac{4f_x}{d}\right) \]  (3.1)
where, \( x \) is a distance of any point from the steel bar’s free end, \( \sigma_x \) is the axial stress of the steel at distance \( x \), \( \sigma_0 \) is bond stress at free end, \( d \) is diameter of the steel bar and \( f \) is a constant value which is equal to \([d\sigma_x]\).

Coates and Yu (1970) conducted a series of pull-out test and used the finite element method to analyse the stress distribution around a cylindrical anchorage. Results demonstrated that the elastic moduli of the bolt (\( E_b \)) and the rock (\( E_c \)) are important factors in the bond stress distribution. \( E_b / E_c \) were analysed in three different ratios of 1:10, 1:1 and 10:1. The bond stress at the free end of the bolt was larger for the smaller ratio. Also, a longer length of the bolt was required for load transformation to the rock surface for the very soft rock (\( E_b / E_c > 10 \)).

Farmer (1975) carried out pull-out tests and developed a theoretical model (Equation 3.2a, 3.2b and 3.2c) for the circular elastic anchor (modulus of elasticity, \( E_b \)) surrounded by an elastic grout (modulus of rigidity, \( G_g \)) confined by a rigid borehole. The thickness of the grout was an important factor in the shear stress.

\[
\tau_x = \frac{\xi_x}{(R_b-r_b)} G_g \quad \text{When } R_b - r_b < r_b \quad (3.2a)
\]

\[
\tau_x = \frac{\xi_x}{r_b \ln\frac{R_b}{r_b}} G_g \quad \text{When } R_b - r_b > r_b \quad (3.2b)
\]

The following equation indicates the shear distribution along a typical resin anchor:

\[
\frac{\tau_x}{\sigma_0} = 0.1 \exp\left(-\frac{0.2x}{r_b}\right) \quad (3.2c)
\]

where, \( \tau_x \) is shear stress in resin annulus, \( \xi_x \) is extension in the bolt, \( r_b \) is radius of bolt, \( x \) is distance along the length of bolt starting at the free end of the grout, \( R_b \) is radius of
the borehole, $G_g$ is the shear modulus of grout, and $\sigma_0$ is axial stress at the free end point.

Experimental pull-out tests in concrete, limestone and chalk were conducted to determine the validity of the theoretical analysis. Tests with a low axial load in concrete showed a good correlation, however the result was not very good in the weaker limestone and chalk because the effect of slippage at the grout-bolt interface was neglected. Also, the model considered the behaviour of the system as an elastic one, which was not a realistic in-situ condition.

Fuller and Cox (1978) conducted pull tests to show the benefits of the mechanical reinforcement for providing self-support of a rock mass. The aim of using reinforcement was to decrease the deflection of the weak region of the rock mass to control parallel and normal displacements of joints. Therefore, mathematical models for three different conditions of the joint subjected to shear, tension and combination of shear and tension were developed. Figure 3-1 shows these three different conditions.

Reinforcement of a joint in shear is shown in equations 3.3a, 3.3b and 3.3c.

\[ \delta_b = \frac{L_s}{2} \left( \frac{1}{\sin(\theta-\alpha)} - \frac{1}{\sin \theta} \right) \]  
(3.3a)

\[ \alpha = \theta - \tan^{-1} \left( \frac{L_s}{\frac{L_s}{\tan \theta} + \delta_s} \right) \]  
(3.3b)

\[ \Delta \tau = \frac{R}{A_j} \left( \cos(\theta - \alpha) + \sin(\theta - \alpha) \tan \phi \right) \]  
(3.3c)

The results showed that the maximum shear resistance occurred when the inclination was between 18° and 20°; however the minimum value was for the angles greater than 60°.
Reinforcement of a joint in tension is shown in equations 3.4a, 3.4b and 3.4c.

\[ \delta_b = \frac{L_n}{2} \left[ \frac{1}{\cos(\theta + \alpha')} - \frac{1}{\cos \theta} \right] \quad (3.4a) \]

\[ \alpha' = \tan^{-1} \left[ \tan \theta + \frac{\delta_n}{L_n} \right] - \theta \quad (3.4b) \]

\[ \Delta \sigma = \frac{R}{A_j} \sin(\theta + \alpha') \quad (3.4c) \]

The maximum of normal restraint was achieved when the inclination was approximately 85°.

Reinforcement of a joint in simultaneous tension and shear is shown in equations 3.5a, 3.5b and 3.5c.

\[ \delta_b = \frac{L_s}{2} \left[ \frac{1}{\sin(\theta - \alpha)} - \frac{1}{\sin \theta} \right] + \frac{\delta_n}{2 \sin(\theta - \alpha)} \quad (3.5a) \]

\[ \alpha = \theta - \tan^{-1} \left[ \frac{L_s + \delta_n}{L_s / \tan \theta + \delta_s} \right] \quad (3.5b) \]

\[ \Delta \tau = \frac{R}{A_j} \cos(\theta - \alpha) \quad (3.5c) \]
Hyett et al. (1996) proposed an analytical model to determine the displacement and load distribution along an un-tensioned fully grouted bolt with specified bond stiffness. Two cases of continues and discontinues distribution of rock displacement were studied. The development of axial load in discontinuous rock was higher. Also, the fully grouted reinforcement was considered more effective in hard rock compared with soft rock. Bonding is a frictional factor and depends on the borehole wall confinement, which is lower in soft rock. Also, excavation decreases the bond stress due to stress changes. Thus, debonding at the cable-grout interface is the reason for failure in soft rock and the cable rupture is the reason for failure in hard rocks.

Shear force due to bond per unit is derived as:

\[ F_s = k(U_r - U_x) \]  

(3.6)
Where; $F_s$ is shear force due to bond per unit, $k$ is stiffness, $U_r$ is the displacement at the tunnel wall, and $U_x$ is distribution of axial displacement along the anchor.

Li and Stillborg (1999) developed three analytical models for rock bolts, which were:

- Model to show the behaviour of bolts exposed to a concentrated pull load in pull-out test,
- Model to show the behaviour of bolts installed in uniformly deformed rock masses, and
- Model to show the behaviour of bolts subjected to the opening of individual rock joints.

The first model was based on laboratory tests while the second and third models were based on the *in-situ* conditions. A series of pull-out test were conducted to prove the models. As Figure 3-2 shows, the results of the pull-out tests and *in-situ* tests were different. The reason for these differences was that bolts in the pull-out test only had anchor length however the length was bigger in *in situ* tests.
Figure 3-2 Distribution of axial stress (a) along a grouted steel bar during a pull-out test, after Hawkes and Evans (Hawkes and Evans, 1951), and (b) along a grouted rock bolts in situ, after Sun (Sun, 1984)

Rock bolts in pull-out test: Figure 3-3 represents a distribution of shear stress along a fully grouted rock bolt, which is subjected to an axial load. From the start point up to $x_0$, the shear stress is equal to zero due to decoupling. From $x_0$ to $x_1$, the shear stress increases to $s_r$ because the bolt interface is partially decoupled. From $x_1$ to $x_2$, the shear stress increases up to its maximum value ($s_p$) and after this point, the shear stress decreases exponentially to the end of the bolt because deformation becomes compatible (Li and Stillborg 1999).
The shear strength of a fully grouted rock bolt is determined by the following equations:

\[
\tau_b = \frac{\alpha}{2} \sigma_{b0} e^{-2\alpha \frac{x}{a_0}} \quad (3.6a)
\]

\[
\alpha^2 = \frac{2G_r G_g}{E_b \left[ G_r \ln\left(\frac{d_b}{d_b}\right) + G_g \ln\left(\frac{d_0}{d_b}\right) \right]} \quad (3.6b)
\]

\[
G_r = \frac{E_r}{2(1+v_r)} \quad (3.6c)
\]

\[
G_g = \frac{E_g}{2(1+v_g)} \quad (3.6d)
\]

The applied load \(P_0\) is equal to the shear force at the interface of the bolt. Therefore:

\[
P_0 = \pi d_b \int_0^L \tau_b \, dx = s \pi d_b x_2 + \frac{\pi d_b^2 s}{2} \left[ 1 - e^{-2\alpha \frac{x_2}{a_0}} \right] \quad (3.7a)
\]

\[
s = \frac{P_{\text{max}}}{\pi d_b L} \quad (3.7b)
\]

where, \(\tau_b\) is shear stress along a fully grouted rock bolt, \(\sigma_{b0}\) is axial stress of the bolt at the loading point, \(d_0\) is diameter of a circle in the rock outside where the influence of the fully grouted rock bolt is determined by the following equations:
the bolt disappears, \(d_g\) is diameter of borehole, \(d_b\) is diameter of bolt, \(E_b\) is Young’s modulus of bolt, \(E_r\) is Young’s modulus of rock mass, \(\nu_r\) is Poisson’s ratio of rock mass, \(\nu_g\) is Poisson’s ratio of grout, and \(P_0\) is applied load (Li and Stillborg 1999).

**Rock bolts in-situ:** The *in situ* situation is different and deformation of rock applies the load to the rock bolt. In this situation, the shear stress of a bolt is calculated by:

\[
\tau_b(x) = \xi G_r \left[ \frac{A}{\pi d_b} \frac{d^2 u_r}{dx^2} - \frac{\alpha}{2} \int_{r_l}^{x} \frac{d^2 u_r}{dt^2} e^{-2a\left(\frac{y}{d_b}\right)} dt \right]
\]

\[
\xi = \frac{2(1+\nu_r)A_i E_b}{AE_b + A_i E_r}
\]

where, \(u_r\) is original radial displacement of the rock at \(x\) (without bolting), and \(A_i\) is influencing area of the bolt in the rock.

A model for bolts subjected to uniform rock deformation: In this situation, both sides of the bolt intersecting the joint are influenced by the tensile load because of opening of the rock joint. When the opening in the rock mass increases, decoupling in the bolt grows, this starts from the interface of the joint and the bolt. The model for the tensile stress of a fully grouted rock bolt and the rock opening is derived from the following equations:

\[
\delta_j \approx \frac{\sigma_{bo} d_b}{\alpha E_b}, \text{ when } \sigma_{bo} \leq \frac{2s_p}{\alpha}
\]

\[
\delta_j = \frac{2}{E_b} \left[ \sigma_{bo} x^2 - 2 \omega s_p \left( \frac{(x - \Delta)^2}{d_b} \right) - (2\omega + 1)s_p \frac{2\Delta^2}{3d_b} + \frac{s_p}{\alpha^2} d_b \right]
\]

where, \(\delta_j\) is opening displacement, and \(\sigma_{bo}\) is the tensile axial stress of the bolt at the joint.
The result from this research showed that the point of loading was the first place that decoupling starts in the bolt which propagated along the bolt by increasing the axial load (Li and Stillborg 1999).

Morsy et al. (2004) conducted a series of pull tests and used a three-dimensional finite element method to simulate interaction between rock and grout using a modified friction model. It was assumed that only slippage, no separation, occurs in the interface between the rock and grout. Test results showed that decoupling mostly occurs between the grout and rock rather than the rock bolt and grout. The model studied the cohesion and friction behaviour of fully grouted bolts before and after any slippage occurred. The proposed mathematical model was based on the Mohr-Coulomb equation:

$$\tau_{crit} = c + \mu \sigma$$ (3.10)

where, $\tau_{crit}$ is the critical shear stress, $c$ is cohesion component at the grout/rock interface, $\mu$ is the coefficient of internal friction for contact surfaces, and $\sigma$ is the applied axial stress.

The surface of the grout was represented using the slave nodes configured as:

$$A_c = \frac{A}{n_n}$$ (3.11)

where, $A$ is an outer surface area and $n_n$ is the number of slave nodes.

Therefore, the critical shear force was measured by:

$$F_{crit} = c \times A_c + \mu f_n$$ (3.12)

where, $f_n$ is a normal force at the contact point estimated at the end of the time increment.
When the slippage occurred, the tangential force was calculated as:

\[ F_t = F_{\text{crit}} - s \times K_p \]  
\[(3.13a)\]

\[ S = \sum ds \]  
\[(3.13b)\]

where, \(s\) is a frictional slip, \(ds\) is an incremental frictional slip, and \(K_p\) is the post-peak shear stiffness of the grout/rock interface.

It was concluded that the axial load distribution along the bolt for a small axial load was exponential but by increasing axial load it became mostly linear (Figure 3-4).

![Figure 3-4](image)

**Figure 3-4** bolt axial stress distribution at; left: small pull out load, right: large pull out load (Morsy et al., 2004)

### 3.3 Mathematical simulations on axial performance of cable bolts by pull-out tests

Yazici and Kaiser (1992) proposed an analytical model to determine the capacity of fully grouted cables. The combination of cable, grout and rock system was simulated by the thick cylinder equation. The model considered rock as a cylinder with infinite outer radius. Four different factors were considered in this method, including axial and lateral
displacements, the pressure at bolt-grout interface and bond strength. The relationship for this model is:

\[
\tau = \sigma \tan \left( t_0 \left[ 1 - \left( \frac{\sigma}{\sigma_{lim}} \right)^{\beta_r} \right] + \emptyset \right)
\]  

(3.14)

where, \( \tau \) is shear or bond strength, \( \sigma \) is radial stress at the bolt-grout interface, \( \emptyset \) is friction angle between the bolt and grout, \( t_0 \) is apparent dilation angle, \( \beta_r \) is reduction coefficient of dilation angle (\( \beta_r = 0.25 \)), and \( \sigma_{lim} \) is limitation stress (compressive strength of grout).

In addition, the equation between axial and lateral displacement is:

\[
U_l = U_{ax} \tan (i)
\]  

(3.15)

where, \( U_l \) is lateral displacement at the bolt-grout interface, and \( U_{ax} \) is axial displacement of the bolt.

Figure 3-5 shows the bond strength model. This model had limitation, because it did not consider the length of the cable and also the interaction between the rock mass and the cable bolt was neglected.
Diederichs et al. (1993) developed an analytical model to evaluate the bond strength of the grouted cable bolt in rock mass. The main reason for failure in the mine is attributed to the loss of frictional bond strength of cable/grout and decreasing the stress during excavation. Important factors in the bond strength of cables were grout, rock stiffness, borehole quality, grout strength and stress change after cable installation.

Results indicated that the grout stiffness is highly related to the ratio of water to cement (w:c) ratio, which may vary between 3:10 and 6:10. The grout stiffness increases by decreasing the w:c ratio. Also, the rock stiffness ($E_r$) is an important factor in the overall stiffness of the system; however it has no effect on the dilation limitation. The crack propagation will be unstable when the ratio of $E_r / E_{\text{grout}}$ is more than 7:10.

**Figure 3-5** Bond Strength Model (BSM) (Yazici and Kaiser, 1992)
The proposed model was a semi-empirical model and the limit of dilation had to be calibrated by test results (140 tests were conducted). The methodology was pull-out test and the failure was caused for two reasons:

1. Rotation of the cable out of the grout
2. Straight axial and shearing at the interface

The dilation mostly occurs due to reason 1 rather than reason 2. The model is:

\[
\sigma_b = \sigma_{b_0} + (1 - \frac{P}{\sigma_g}) \frac{\beta_b}{\varepsilon_g}
\] (3.16)

Where, \(\sigma_b\) is bond strength, \(\sigma_{b_0}\) is dilation axis by an intercept, the absolute limit of dilation in the absence of any external confinement, \(P\) is pressure, \(\sigma_g\) is Uniaxial Compressive Strength of the grout, and \(\beta_b\) is bounding functions (empirical constant values of \(\beta_{upper}\) and \(\beta_{lower}\)).

Dube (1996) pull-out tested standard and modified (nutcase, Garford bulb and birdcage) cable bolts, at various orientation to the direction of pull. The finite element program ANSYS 4.4a for numerical simulation. The cable was fixed by using barrel and wedge in the anchor section. The result revealed that; with higher radial confinement and lower w/c ratios the standard cables reached higher bond capacities. The bond capacity of modified cables where higher compared with standard ones. both types of cable bolts were pulled out due to frictional loss.

The higher bond capacity in standard cable bolts was achieved when the component of the shear increased (exceeding 30° off pure tension) as a result of cable bending, grout crushing and higher friction in front of the cable bolt with the cable became significantly softer. Thus, the modes of failure were different for low and high angles of pulling. Failure in low angle was due to frictional pull out however in high angle led to
cable rupture. On the other hand, the bond capacity of modified cable bolts decreased when the component of shear increased because the cable was subjected to greater shear and bending.

Bouteldja (2000) presented a numerical model to analyse the performance of cable bolts. Deformation and distribution of loads are important factors in the safe design of cable bolts subjected to pull testing. There are many other important factors in the cable bolt design, such as type of cable, tensioning, anchoring, properties of rock and mining induced stress. The model of the shear bond stiffness \( k \) is a function of the cable type, modulus of elasticity, confining pressure variation and water to cement ratio.

\[
K = f(\Delta\sigma). f(E). f(w:c) \tag{3.17a}
\]

\( K = K_0 \), when the variation of confining pressure is equal to zero \( (\Delta\sigma = 0 \text{MPa}) \)

The relationship between the shear bond stiffness \( k \) and Modulus of elasticity \( E \) is linear based on the following equation:

\[
K_0 = a_0 E_m + b_0 \tag{3.17b}
\]

where, \( K_0 \) is the shear bond stiffness for \( \Delta\sigma = 0 \) (MN/m/m), \( a_0 \) and \( b_0 \) are coefficients depending on the cable geometry and water: cement ratio, and \( E_m \) is the modulus of elasticity of host medium (GPa).

Figure 3-6 shows the anticipated load distribution along the tensioned cable bolts. Results from this figure show that by tensioning the cable bolt, axial load increases its potential strength. In general, the resin grout is more suitable if compared with cement grout because of its hardening properties and its higher strength over the bolt length.
Mousavi et al. (2002) conducted pull-out tests to study the bond failure mechanism of conventional and modified geometry cable bolts by determination of the axial load and the radial dilation due to the axial displacement of the cable under constant confining pressure (constant normal stiffness) using the conventional Hoek cell. The result revealed that the bulge structure in modified cable bolts provided higher axial load and radial dilation. The higher radial pressure was the result of higher dilation and the degree of rock mass confinement. The main finding was that the bond failure occurred in conventional cable bolts however, the grout shear was the reason for failure in
modified cable bolts. An analytical model was proposed to measure the bond capacity of cable bolt including two components of incremental slip and incremental mining-induced stress change. The second component is negligible in laboratory testing conducted in confining pipe.

\[ dF_p = K_d d(u_r - u_x) + K_s d\sigma \]  \hspace{1cm} (3.18)

Where; \( dF_p \) is the incremental change in bond capacity, \( K \) is stiffness, \( u_r \) is rock displacement, \( u_x \) is bolt displacement, \( d\sigma \) is the stress change along the cable bolt axis.

Wu et al. (2009) proposed an analytical model to simulate the behaviour of rock tendons. Figure 3-7 shows the tri-linear model which consists of shear bond stress versus slip. It consists of three stages in behaviour. The first phase (I) defines the linear relationship. The second phase (II) describes the ultimate load and debonding that occurs due to the reduction in stress. The residual stress occurs in the third phase (III). The proposed model is:

\[ \tau = ms + f \]  \hspace{1cm} (3.18)

where, \( \tau \) is shear stress on the tendon-grout interface, \( s \) is slip between the tendon and grout, \( m \) and \( f \) are the coefficients that are derived from Figure 3-7.

\[ 0 \leq s \leq s_1 \quad m = m_1 = \frac{t_1}{s_1}, \quad f = 0 \]  \hspace{1cm} (3.19a)

\[ s_1 \leq s \leq s_2 \quad m = m_2 = \frac{t_1 - t_2}{s_1 - s_2}, \quad f = \frac{t_2 s_1 - t_1 s_2}{s_1 - s_2} \]  \hspace{1cm} (3.19b)

\[ s_2 \leq s \quad m = 0, \quad f = t_2 \]  \hspace{1cm} (3.19c)
Chen et al. (2015) used the model proposed by Wu et al. (2009) to report on the load transfer behaviour of fully grouted cable bolts by using a tri-linear model for elastic, elastic-softening, elastic-softening-debonding, pure softening, softening-debonding and debonding stages.

- Elastic stage:

Axial and shear stress on the cable/grout interface:

\[
\sigma(x) = \frac{4F}{\pi D^2} e^{ax} - e^{-ax} \tag{3.20a}
\]

\[
\tau(x) = \frac{P}{\pi D} e^{ax} - e^{-ax} \tag{3.20b}
\]

\[
\alpha^2 = \frac{4kE_b}{D(E_b+Dk(1+v_b)) \ln \frac{D+2r_b}{D}} \left[ \frac{1}{E_c} + \frac{D^2}{4E_m b(D+b+2r_b)} \right] \tag{3.20c}
\]

where, \(\sigma(x)\) is axial stress at point \(x\), \(\tau(x)\) is shear stress at point \(x\), \(D\) is cable diameter, \(E_c\) is the Young’s modulus of cable bolts, \(E_m\) is Young’s modulus of the confining
medium, \( t_g \) is the thickness of grout, \( L_c \) is the embedment length of cable bolt and \( F \) is axial load at the loaded end.

- **Elastic-softening stage:**

\[
\sigma_b(x) = \left[ \frac{4\tau_p}{D\alpha} \tan h(L_c - a_s) \right] \cos \left( \omega(L_c - a_s) \right) - \frac{4\tau_p}{D\omega} \sin \left( \omega(L_c - a_s) \right) \cos(\omega x) + \left[ \frac{4\tau_p}{D\alpha} \tan h(L_c - a_s) \right] \sin \left( \omega(L_c - a_s) \right) - \frac{4\tau_p}{D\omega} \cos \left( \omega(L_c - a_s) \right) \sin(\omega x) \quad (3.21a)
\]

\[
\tau(x) = -\tau_p \left[ \frac{\omega}{\alpha} \tan h(\alpha(L_c - a_s)) \cos \left( \omega(L_c - a_s) \right) - \sin \left( \omega(L_c - a_s) \right) \right] \sin(\omega x) + \tau_p \left[ \frac{\omega}{\alpha} \tan h(\alpha(L_c - a_s)) \sin \left( \omega(L_c - a_s) \right) + \cos \left( \omega(L_c - a_s) \right) \right] \cos(\omega x) \quad (3.21b)
\]

\[
\omega^2 = \frac{4E_b(\tau_p - \tau_f)}{D E_b(\delta - \delta_t) + D^2(\tau_p - \tau_f)(1 + \frac{v}{2}) \ln \frac{D^2}{D^2 + 2t_g} \left[ \frac{1}{E_b} + \frac{D^2}{4E_m b(D + b + 2t_g)} \right]} \quad (3.21c)
\]

where, \( a_s \) is softening length and \( a_f \) is frictional length.

- **Elastic-softening-debonding stage:**

This stage consisted of the maximum pull-out load. Figure 3-8 shows the softening and frictional lengths along the cable/grout interface.

\[
\sigma_b(x) = \left[ \frac{4\tau_p}{D\alpha} \tan h(L_c - a_s - a_f) \right] \cos \left( \omega(L_c - a_s - a_f) \right) - \frac{4\tau_p}{D\omega} \sin \left( \omega(L_c - a_s - a_f) \right) + \left[ \frac{4\tau_p}{D\alpha} \tan h(L_c - a_s - a_f) \right] \sin \left( \omega(L_c - a_s - a_f) \right) - \frac{4\tau_p}{D\omega} \cos \left( \omega(L_c - a_s - a_f) \right) \sin(\omega x) + \left[ \frac{4\tau_p}{D\alpha} \tan h(L_c - a_s - a_f) \right] \cos \left( \omega(L_c - a_s - a_f) \right) - \frac{4\tau_p}{D\omega} \sin \left( \omega(L_c - a_s - a_f) \right) \cos(\omega x) \quad (3.22a)
\]

\[
\tau(x) = -\tau_p \left[ \frac{\omega}{\alpha} \tan h(\alpha(L_c - a_s - a_f)) \cos \left( \omega(L_c - a_s - a_f) \right) - \sin \left( \omega(L_c - a_s - a_f) \right) \right] \sin(\omega x) + \tau_p \left[ \frac{\omega}{\alpha} \tan h(\alpha(L_c - a_s - a_f)) \sin \left( \omega(L_c - a_s - a_f) \right) + \cos \left( \omega(L_c - a_s - a_f) \right) \right] \cos(\omega x) \quad (3.22b)
\]
Figure 3-8 Softening length and frictional length propagation along the cable/grout interface (Chen et al., 2015)

- Softening-debonding stage:

\[ \tau(x) = \tau_f \frac{\cos(\omega x)}{\cos(\omega a_s)} \]  

(3.23a)

- Debonding stage

\[ \tau(x) = \tau_f \]  

(3.23b)

This model, which was originally proposed by Wu et al. (2009) is capable of simulating the performance of cable bolts from the elastic stage up to debonding.

3.4 Mathematical simulations on shear performance of rock bolts

Tendons are installed to increase strength of the joints, weakness planes and fractures in rock. Tendons have the ability to provide both the axial and shear reinforcement of rock strata (Figure 3-9). This part of the review consists of the tendons shear behaviour looking at single and double shear test methodologies.


**Figure 3- 9** Stability issues in rock mass reinforced by fully grouted bolts (Jalalifar 2006)


Dulacska (1972) carried out a series of single shear tests to examine the action of dowels in cracked concrete to determine the theoretical load-deformation relationships. The steel bars across the cracks and the friction across the concrete surfaces counteract the slipping of the cracks. Figure 3-10 shows the specimen designed for a single shear test. The crack was simulated by two embedded sheet brass layers with thickness of 0.2 mm in the test specimens which were connected in the middle by a skewed steel stirrup.
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Figure 3-10 Details of test specimen construction (Dulacska, 1972)
15 tests were carried out using different concrete grades, three steel sizes and four
different angles for the stirrup. The shear force of bolt was determined by:

\[ S_b = 0.2 d_b^2 \sigma_y \sin \epsilon \left[ 1 + \frac{\sigma_c}{0.03 \sigma_y \sin^2 \epsilon} - 1 \right] \]  (3.24)

where, \( S_b \) is the shear force carried by the bolt, \( d_b \) is the bolt diameter, \( \sigma_c \) is the UCS of
rock, \( \sigma_y \) is the yield stress of bolt, and \( \epsilon \) is the angle of stirrups.

The result from this research showed that failure mostly occurs due to plastic
deformation. Therefore, the slip value was determined from the following equation:

\[ \Delta = \frac{3T}{d_b 10^6} \left( \frac{1}{\sigma_c} \tan \left( \frac{T}{T_f} \right) \right) \]  (3.25)

where, \( T_f \) is failure force and \( \Delta \) is the value of slip.

Bjurstrom (1974) conducted single shear tests to study the shear force transfer in un-
tensioned grouted bolted joints. Granite blocks with natural and artificial joints with
different normal pressures were tested. The angle between the bolt and joints was
variable. The joint surface was smooth and the shear displacement was less than 50 mm.
50 tests were conducted without bolt to verify the joint surface friction and 60 shear
tests were carried out with the bolt installation. The shear displacement was measured
by Linear Variable Differential Transformer (LVDT). As shown in Figure 3-11, the
shear strength depends on three factors: friction, tension in the bolt and the dowel effect
of the bolt. Results showed that failure occurs in tension when the angle between the
bolt and the joint is less than 35°, and failure was a combination of shear and tension by
increasing this angle to more than 40-45°.
As mentioned previously, shear strength has three factors which can be described by the following equations:

Reinforcement effect: \[ S_r = p(\cos \beta + \sin \beta \varphi) \] (3.26a)

Dowel effect: \[ S_d = 0.67d_b^2 \sqrt{\sigma_c \sigma_y} \] (3.26b)

Joint friction: \[ T_j = A_j \sigma_n \tan \varphi \] (3.26c)

where, \( p \) is the axial load corresponding to the yield strength due to shear displacement, \( \beta \) is the initial angle between bolt and normal to the joint, \( \varphi \) is the friction angle of the joint, \( d_b \) is the bolt diameter, \( \sigma_y \) is the bolt yield strength, \( \sigma_c \) is the UCS of the rock, \( A_j \) is the joint area and \( \sigma_n \) is normal stress on joint.
Haas (1976) conducted a series of single shear tests on blocks of chalk and limestone. There were different variables in this research including bolt type, normal pressure on the interfaces and different orientations of the bolt to the shear surface (0°, +45° and -45°) as shown in Figure 3-12. It was observed that applying 170 kPa of normal compressive stress on the slip plane and also orienting the bolt in the direction to the shear progress increased the shear resistance. There was no benefit in pre-tensioning bolts. The issue with the single shear test was non-equilibrium load distribution along the joint plane (Figure 3-13).

**Figure 3-12** Arrangement for bolt shearing testing (Haas, 1976)

**Figure 3-13** Non-equilibrium in vicinity of the shear joint (Haas, 1976)
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Haas (1976) also proposed a mathematical model for calculating the shear resistance of bolted rock. The initial average shear stress was calculated by:

$$\tau_{ave} = \tau_0 + \frac{\mu T_0 \cos \theta_o + T_0 \sin \theta_o}{A_s} \quad (3.27)$$

where, $\tau_{ave}$ is the initial average shear stress with the bolt for impending movement, $\tau_0$ is the average shear stress on the interface without a bolt, $\mu$ is the coefficient of friction, $\theta_o$ is the initial orientation of bolt, $A_s$ is the area in shear, and $T_0$ is tension in the bolt.

As Figure 3-14 shows, by increasing the shear displacement to value d, the distance from the anchor to the head changed from $L_0$ to L. By increasing the length, the strain over the length of the bolt increased and the load in the bolt increased to the yield limit. This remained constant after the elastic limit. The average shear stress in this stage was calculated by:

$$\tau_{ave} = \tau_0 + \frac{\mu T \cos \theta_1 + T \sin \theta_1}{A_s} \quad (3.28a)$$

$$L = \sqrt{d^2 + 2dL_0 \sin \theta + L_0^2} \quad (3.28b)$$

$$\theta_1 = \arctan\left(\frac{d + L_0 \sin \theta}{L_0 \cos \theta}\right) \quad (3.28c)$$

$$T = E_b A_b (\varepsilon_0 + \varepsilon_1) \quad \text{for} \quad (\varepsilon_0 + \varepsilon_1) < \varepsilon_y \quad (3.28d)$$

$$T = \text{Yield Strength} \quad \text{for} \quad (\varepsilon_0 + \varepsilon_1) \geq \varepsilon_y \quad (3.28e)$$

where, $T$ is bolt tension at displacement d, $\theta_1$ is bolt orientation at displacement d, $A_s$ is the area in shear ($D^*(D-d)$), $D$ is the height and depth of the shear block, $E_b$ is the Young’s modulus of the bolt, $A_b$ is the cross-sectional area of the bolt, $\varepsilon_0$ is the axial
strain induced by the initial torque applied to the bolt, and $\varepsilon_1$ is the increment of axial strain induced by shear displacement lead in to equation 3.29.

$$d = \frac{(L - L_0)}{L_0} \quad (3.29)$$

where, $\varepsilon_y$ is strain at the onset of yielding.

**Figure 3-14** Geometry of conventional bolt subjected to shear displacement

Azuar (1977) studied resin-grouted bolts embedded in concrete by conducting laboratory tests. Shear contribution of rock bolt installed perpendicular to the joint shear plane was between 60% and 80% of the bolt’s ultimate tensile load however this value increased up to 90% for the inclined bolts. Thus, when the bolts were installed perpendicular to the joint, they did not experience considerable tension. Moreover, the bolt contribution to shear resistance was separable from the friction characteristics of the joint.

Hibino and Motojima (1981) studied performance of un-grouted bolts installed in concrete blocks subjected to shearing. No difference was observed due to the bolt
inclination which was in contrast with results from other researchers. Also, it was concluded that pre-tensioning does not affect the shear resistance of bolts although it decreases the shear displacement at peak shear load.

Dight (1982) studied the performance of fully grouted rock bolts in shearing. The point of flexure and bar response to shearing is important factors in studying shear behaviour of rock bolts. The bar response to shearing was expressed by the following equation:

\[ F_b = l \cdot D \cdot P_u \]  

(3.30)

where, \( F_b \) is the bar response to shearing, \( l \) is the distance to maximum moment for shear plane, \( D \) (\( D_b \)) is the diameter of the bar when grout is weaker than rock, \( D \) (\( D_{bh} \)) is the diameter of the grout when grout is stronger than the rock, and \( P_u \) is the bearing capacity of rock or grout.

Spang and Egger (1990) conducted a series of single shear tests on fully grouted untensioned bolts in jointed rock and used Finite Element Method for a numerical three-dimensional model simulation. Bolt inclination was 90° and 30° to the joint surface. Three stages of behaviour for the fully grouted bolts in the jointed rock mass were considered as elastic, yield and plastic stages. The behaviour in the elastic zone followed the Mohr-Coulomb relationship. It was monitored that the yielding stage in the bolt and mortar occurs at the very stage of loading when the shear displacement was 1 mm and the bolt only reached 10% of its ultimate strength. Therefore, plastic zone governed the main aspect of performance. The ADINA code from the Swiss Federal Institute of Technology, Lausanne was used for the numerical simulation. The Drucker-Prager and von Misses failure criteria were used to define the elastic-plastic behaviour of materials.
The model was simplified without considering the joint-friction of blocks and also that blocks were held together by the grouted bolt. Results showed a higher shear resistance for the inclined bolt compared with the perpendicular one. Also, the deformation amount in the inclined bolt was less than the bolt without inclination. The failure in the perpendicular bolt was the result of bending with a combination of shear and tension across the joint surface; however failure of the inclined bolt was only due to the tension near the shear surface. The plastic strain in the inclined bolt (30°) was less than the perpendicular one as shown in Figure 3-15. The created gap under the shear force and the bond failure between steel and mortar for the perpendicular bolt was more than the inclined one (Figure 3-16).

**Figure 3-15** Plastic strains in the bolt for T=30 kN and (α = 0° and 30°) (Spang and Egger, 1990)
Figure 3-16 Gap between bolt and mortar for $\alpha = 0^\circ$ (T = 25 kN) and $30^\circ$ (T = 25, 35 kN) (Spang and Egger, 1990)

An analytical model was proposed to calculate the shear resistance of a bolted joint and the corresponding shear displacement, as:

$$S_o = P_t (1.0 + \Delta T_{A+G})m_F \cdot m_R$$

(3.31a)

$$m_R = (0.85 + 0.45 \tan \emptyset)$$

(3.31b)

$$m_F = 1.55EM^{-0.2}$$

(3.31c)

Or

$$m_F = 1.55.\sigma_c^{-0.14}$$

(3.32a)

$$\Delta T_{A+G} = 0.007. EM^{1.5}.sin^2(\alpha+i)$$

(3.32b)

Or

$$\Delta T_{A+G} = 0.007. \sigma_c^{1.07}.sin^2(\alpha+i)$$

(3.33a)

$$S_o = P_t \left[1.55 + 0.011. EM^{1.5}.sin^2(\theta+i)\right]EM^{-0.2}.(0.85 + 0.45 \tan \emptyset)$$

(3.33b)
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Or

\[ S_\sigma = P_t \cdot \left[ 1.55 + 0.011 \cdot \sigma_c^{1.07} \cdot \sin^2(\theta_\pm) \right] \cdot \sigma_c^{-0.14} \cdot (0.85 + 0.45 \tan \theta) \]  \hspace{1cm} (3.33c)

\[ v = (15.2 - 55.2 \cdot EM^{-0.2} + 56.2 \cdot EM^{-0.4}) \cdot (1 - \tan \theta \cdot \left( \frac{20}{EM} \right)^{0.25} \cdot (\cos \theta)^{-0.5}) \]  \hspace{1cm} (3.33d)

Or

\[ v = (15.2 - 55.2 \cdot \sigma_c^{-0.14} + 56.2 \cdot \sigma_c^{-0.28}) \cdot (1 - \tan \theta \cdot \left( \frac{70}{\sigma_c} \right)^{0.125} \cdot (\cos \theta)^{-0.5}) \]  \hspace{1cm} (3.33e)

where, \( S_\sigma \) is the maximum shear resistance, \( P_t \) is the maximum tensile load of the bolt, \( \Delta T_{\alpha+c} \) is an expression for the bolt inclination, \( m_F \) is an influence of rock deformability on the shear resistance of the bolted joint to the maximum tensile load of the bolt, \( m_R \) is a dimensionless value, which shows the influence of the friction of joint on the shearing resistance of the bolted joint in relation to the maximum value of tensile load, \( EM \) is the penetration modulus, \( \phi \) is the angle of friction, \( f(s) \) is shear displacement, and \( \theta \) is the bolt’s inclination.

Aydan and Kawamoto (1992) studied the effect of rock bolts on discontinuous rock mass in shear reinforcement. The instabilities in the structure mostly depend on failures in shear, sliding, bending, buckling, and flexural or block toppling failure and block falls. The main parameter to resist axial and shear loads are the steel bar within the rock bolt system. These factors depend on the inclination of bolts, discontinuous surface characteristics, rock element properties and the interaction between the steel bar and ground. Thus, to model the rock bolt in axial and shear load, it was necessary to consider the steel bar and the steel surroundings (Aydan, 1989, Aydan et al., 1985). Therefore, the stiffness of bolt element was measured by:
Moreover, the required reinforcement for the slab stability with discontinuity at various inclination angles was estimated using:

\[
\sum_{i=1}^{n} \sigma_s = \frac{W_g}{A_b} [\sin \alpha - \cos \alpha \tan \varphi]
\]  

(3.35)

where, \(W_g\) is the weight of sliding mass.

Pellet and Egger (1996) developed an analytical model to investigate the shear strength of an un-tensioned fully grouted rock bolt. The bolt’s behaviour and interaction between the axial and shear forces and its large plastic displacements were measured. The behaviour of rock bolts in the elastic and plastic zones were studied as shown in Figure 3-17. Also, the bolt’s elastic and plastic deformations are presented in Figure 3-18. The Tresca criterion was used to model the bolt in failure during the direct shear test where the combination of shear and axial loads were experienced.
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Figure 3-17 Force components and deformation of a bolt in (a) elastic zone, and (b) plastic zone (Pellet and Egger, 1996)

![Figure 3-17](image)

Figure 3-18 Shear force versus axial force in the bolt (a) Elastic zone, (b) Plastic zone (Pellet and Egger, 1996)

![Figure 3-18](image)

Shear force at point O in Figure 3-16 and shear plane were mathematically expressed as:
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\[ S_y = \frac{1}{2} P_u D_b \left( \frac{\pi D_b^2 \sigma_y}{4} - N_{oe} \right) \]  
(3.36a)

\[ S_f = \frac{\pi D_b^2}{8} \sigma_{ec} \sqrt{1 - 16 \left( \frac{N_{of}}{\pi D_b^2 \sigma_{ec}} \right)^2} \]  
(3.36b)

\[ U_{oe} = \frac{8192 Q^2_{oe} b}{E_b P_u \pi^4 D_b^2 \sin \beta} \]  
(3.36c)

\[ \Delta U_{op} = \frac{Q_{oe} \sin \omega_{op}}{P_u \sin (\beta - \Delta \omega_{op})} \]  
(3.36d)

\[ u_0 = \frac{24 N_0 Q_0}{E_b P_u \pi^2 D_b^2} \]  
(3.36e)

\[ v_0 = \frac{8192 Q^2_{of} b}{E_b \pi^4 P_u^2 D_b^2} \]  
(3.36f)

\[ U_{of} = U_{oe} + \Delta U_{op} \]  
(3.36g)

where, \( S_y \) is the shear force acting at point O at the yield stress of the bolt, \( P_u \) is the bearing capacity of the grout or rock, \( D_b \) is the bolt diameter, \( \sigma_y \) is the bolt yield stress, \( N_{oe} \) is the axial bolt force at the shear plane when the bolt yields, \( S_f \) is the shear force in bolt at the shear plane during bolt failure, \( \sigma_{ec} \) is the stress in bolt when bolt failure occurs, \( N_{of} \) is the axial bolt force acting at the shear plane during bolt failure, \( U_{oe} \) is the axial displacement at point O, and \( E_b \) is bolt elasticity.

\[ \beta = \tan^{-1} \frac{v_0}{u_0} \]  
(3.37)

where, \( v_0 \) is shear displacement at point O and \( U_{tot} \) is the total displacement.

Finally, the increment of plastic rotation was calculated by:

\[ \Delta \omega_{op} = \arccos \left[ \frac{1}{l_f} \sin^2 \beta \pm \sqrt{\cos^2 \beta \left( 1 - \left( \frac{l_e}{l_f} \right)^2 \sin^2 \beta \right)} \right] \]  
(3.38)
where, \( l_e \) is the distance between point O and Point A, and \( l_f \) is the length of the part O-A at failure.

The angle between the bolt and joint has a significant impact on the maximum joint displacement. Displacement dropped rapidly by decreasing the inclination angle of bolt and joint (see Figure 3-19).

![Figure 3-19 Joint displacement as a function of angle β for different UCS (Pellet and Egger, 1996)](image)

Chappell (1987) determined that prestressing the rock bolt before grouting increases the normal and shear stiffness. The effect of stress on rock bolts in soft and hard rocks are different. The joint normal stiffness increases by a factor of 10 and 2 for soft and hard rocks respectively when a rock bolt is prestressed to 10 t. It was observed that the effect of rock bolting on rock mass stiffness and anisotropy was more on single joint rather than a multiple ones. A rock bolt not only affected the joint stiffness and strength characteristics, but it also affected the deformation that controls the rock mass poison’s ratio (Chappell 1989).
Ferrero (1995) studied the mechanical behaviour of reinforced discontinuous rock joint. The main focus of this research was on the effects of steel type, bar diameter, reinforcement type (bars and tubes) and rock material type. Figure 3-20 shows the laboratory test device (shear box system). Shear load, shear displacement, axial force acting on a bolt and the bar strain at a certain number of sections were recorded during tests. The rock was cut to specific size as square sections of 300 mm side, height of 400 mm and a global height of 800 mm. The contact surfaces were completely smooth to avoid joint roughness. Prestress load was applied to the bolts before pouring grout into the system.

The experimental results showed that the shear resistance was proportional to the square of the bolt diameter. The highest level of shear resistance was achieved for the most ductile steel reinforcement, which reached the ultimate tensile resistance of the material at failure. Stiffer rock material provided a higher shear stress in the bars but a lower global resistance of reinforced joint. Pretension appeared not to affect the maximum resistance; however it affected the shear displacement.

![Figure 3-20](image_url)  
*Figure 3-20* Laboratory test device (Ferrero, 1995)
The global joint resistance was given as:

\[ F_g = T_r \cos i - S_d \sin i - (T_r \sin i - S_d \cos i) \tan \varphi \]  

(3.39)

where, \( F_g \) is the global reinforced joint resistance, \( S_d \) is the dowel effect (shear force induced in the bar), \( T_r \) is the load induced in the bar, \( i \) is the angle between the joint and the dowel axis and \( \varphi \) is the joint friction angle.

Two different types of failure were studied based on the strong and stiff rock material \((\sigma_c > 50 \text{ MPa})\) and a weaker one \((\sigma_c < 50 \text{ MPa})\). The shear resistance of the system depended mainly on the yielding steel although the shear resistance increased due to the higher strength and stiffer rock materials. The first mode of failure was based on the combination of axial and shear force acting at the bar-joint intersection (Figure 3-21) however the second one was an axial failure due to reaching the ultimate tensile load or ultimate steel strain after formation of two plastic hinges (Figure 3-22). The failure occurred in pull-out mechanics when the rock mass was too weak.

Plasticity occurred in a combination of tension and bending and was evaluated by:

\[ \left( \frac{T_s}{T_y} \right)^2 + \frac{M_0}{M_y} = 1 \]  

(3.40a)

\[ M_y = 1.7 \sigma_y \left( \frac{D_b^3 \pi}{32} \right) \]  

(3.40b)

where, \( \sigma_y \) is the steel yield stress, \( D_b \) is the bar diameter and \( M_y \) is the plastic moment.
In the first failure mechanism, O was the bar-joint intersection. A was the point of maximum moment where the shear force was equal to zero. The curvature of the bar and tension force at the bar-joint intersections was determined by following equations:

\[ k = \frac{\frac{d^2y}{dx^2}}{(1 + \left(\frac{d^2x}{dy^2}\right)^{3/2})^{3/2}} = \frac{2y_0}{x_0^2} \left(\frac{4y_0^2}{x_0^2}\right)^{3/2} \]  \hspace{1cm} (3.41a)
The bar force along the bar length for the second mechanism of failure was determined by the following equation:

\[ T_r = \mu p_u D_b s \]  

(3.42)

The result indicated a good agreement with experimental data and analytical models.

Afridi et al. (2001) studied the shear resistance of both intact and jointed rock by a DR direct shear test machine (Figure 3-23). The study showed that by applying shear force along a shear plane, the shear stress increased before peak shear strength with small changes in the shear and normal displacement. The peak shear strength occurred in the first stages of the test with a great contribution to the shear resistance of the plane.

![Figure 3-23 Schematic drawing of shear machine (Afridi et al., 2001)](image)

Aziz (2002) proposed a new technique to determine the load transfer capacity of resin anchored bolts. One suggested method of determining the bolt load transfer and the shear stress developed at the bolt-resin interface was by using strain gauged instrumented rock bolts in underground. The following equation (Fuller and Cox, 1975;
Gale, 1986; Fabjanczyk and Tarrant, 1992, and Signer et al., 1997) describes the shear stress at the bolt-resin interface:

\[ \Delta \tau = \frac{F_1 - F_2}{\pi D_b l} \]  

(3.43)

where, \( \Delta \tau \) is shear stress at bolt-resin interface, \( F_1 \) is the axial force acting on the bolt at strain gauge position 1 (calculated from strain gauge reading), \( F_2 \) is the axial force acting on the bolt at strain gauge position 2 (calculated from strain gauge reading), \( D_b \) is the bolt diameter and \( l \) is the distance between strain gauge positions 1 and 2.

The stiffness of the resin/rock system was measured based on the following equation developed from experimental study:

\[ \sigma_n = \sigma_{n0} + \frac{k_n \delta_v}{A_b} \]  

(3.44)

where, \( \sigma_n \) is the effective normal stress, \( \sigma_{n0} \) is the initial normal stress, \( k_n \) is the system stiffness, \( \delta_v \) is the vertical displacement (dilation) and \( A_b \) is the bolt surface area.

Thus, the effective normal stress was higher for bolt type I due to deeper and wider spaced rib profile in that bolt type.

Grasselli (2005) conducted a series of double shear tests on fully grouted solid and Swellex bolts to compare their performance in shear. Three concrete blocks of the same size measuring 1000 x 600 x 600 mm with smooth joints were used (Figure 3-24). Concrete blocks had the strength of 35 MPa, elastic modulus of 29.7 GPa and the friction angle of 40°. The grout had 50 MPa strength and 13.2 GPa of elastic modulus.
The shear test was conducted by applying the load to the middle block and pushing it down. The shear load, the normal force to the joint, vertical displacement of the middle block and the bolt deformation were measured. The dimensionless contribution of bolt to the shear strength allowed comparing the effect of different types of bolts with the shear strength of the system in a given angle of bolt installation across joints. The contribution of the bolts to shear strength was calculated using:

\[
S^* = \frac{bolts \text{ contribution}}{2nF_{max}} = \frac{T_V - 2N_j \tan \varphi_l}{2nF_{max}}
\]  

(3.45)

where, \( S^* \) is the dimensionless bolt contribution to shear strength, \( T_V \) is the difference between the applied shear force, \( N_j \) is Frictional strength provided by the unbolted smooth joints, \( F_{max} \) is the ultimate tensile load of the bolt, \( \varphi_l \) is the frictional angle of the smooth joint, and \( n \) is the number of bolt sections on each joint.
Two plastic symmetrical hinges appeared at the rock joints when testing the fully grouted bolt followed by the expanding plastic deformation zone until failure. Swellex bolts performed differently due to lack of the grout, starting to yield plastically near the joint plane. Two plastic zones formed at the inner surface of the tube. Thus, a Swellex bolt had a higher section deformation although it had a lower shear resistance. The double shear system was unconfined which was not realistic and different to the field conditions.

Jalalifar et al. (2006) studied the effect of surface profile of rock bolt, rock strength and pretension load on the bending behaviour of fully grouted bolts subjected to double shear test. Results showed that the bolts gradually deformed under shear load. The resultant loads in the bolt consisted of two components; the axial load, $N$, and a lateral shear load, $Q$, at the bolt-joint intersection. The bolt-joint intersection had zero bending moment and the hinge point had the maximum bending moment and zero shear stress. Distribution of normal stresses was produced by the bolt axial force and the bending moment, as:

$$
\sigma = \frac{N}{A} - \frac{My}{I}
$$

(3.46)

where, $\sigma$ is the normal stress action on the bolt, $N$ is the axial force acting on the bolt, $A$ is the cross-sectional area of the bolt, $M$ is the bending moment, $y$ is the distance from the neutral axis and $I$ is the second moment of inertia.

The maximum bending moment at point D in the elastic region occurred due to the lateral load and the curvature in the deflected beam as shown in Figure 3-25. The moment was measured as:

$$
M_D = \frac{Q^2r}{2P(x)}
$$

(3.47)
where, $M_D$ is the bending moment at point D, $P(x) = f(K_y, U_y)$ is the reaction force by subgrade, $Q_{cf}$ is the shear load, $K_y$ is the medium stiffness and $U_y$ is the displacement along the joint.

![Figure 3-25](image)

**Figure 3-25** Applied and induced loads in bolt (Jalalifar et al., 2006)

Results showed higher shear resistance of bolts in 40 MPa concrete blocks compared with 20 MPa strength concrete blocks. The higher initial pretension load provided a higher shear load at the limit of maximum frictional bonding strength for the majority of cases. The 3D numerical analysis by ANSYS version 8.0 confirmed the experimental results.

### 3.5 Mathematical simulations on shear performance of cable bolts

Aziz et al. (2015b) proposed a mathematical model to determine the peak shear load of the pre-tensioned fully grouted cable bolts subjected to double shear tests using the combination of Mohr-Coulomb Criterion and Fourier series scheme, as:

$$\tau - \sigma \tan(\varphi) - c = 0$$

(2.48a)

where, $\tau$ is shear stress, $\sigma$ is normal stress, $\varphi$ is the friction angle, and $c$ is the cohesion.
\[ \sigma = \frac{a_0}{2} + \sum_{n=1}^{\infty} \left[ a_n \cos \left( \frac{2n\pi u}{L_s} \right) + b_n \sin \left( \frac{2n\pi u}{L_s} \right) \right] \]  
\hspace{2cm} (2.48b)

\[ a_n = \frac{2}{L_s} \int_{0}^{L_s} \sigma_n \cos \left( \frac{2n\pi u}{L_s} \right) du \]  
\hspace{2cm} (2.48c)

\[ b_n = \frac{2}{L_s} \int_{0}^{L_s} \sigma_n \sin \left( \frac{2n\pi u}{L_s} \right) du \]  
\hspace{2cm} (2.48d)

This model was only capable of determining the peak shear strength of the cable bolt subjected to double shearing.

The proposed model was:

\[ \tau_p = \left( \frac{a_0}{2} + \sum_{n=1}^{3} [a_n \cos (\frac{2n\pi u}{L_s})] \right) \tan \varphi + c \]  
\hspace{2cm} (2.49a)

\[ \frac{d(\frac{a_0}{2} + \sum_{n=1}^{3} [a_n \cos (\frac{2n\pi u}{L_s})]) \tan \varphi + c}{du} = 0 \]  
\hspace{2cm} (2.49b)

\[ \tau_p = \left( \frac{a_0}{2} + \sum_{n=1}^{3} [a_n \cos (n \cos^{-1} \left( \frac{-4a_2 + \sqrt{(16-48a_1 a_3 + 144a_3^2)}}{24a_3} \right)) \tan(\varphi) + c \]  
\hspace{2cm} (2.49c)

where, \( \tau_p \) is the shear stress, \( c \) is cohesion, \( a_n \) is Fourier coefficient, \( n \) is the number of Fourier coefficients, which ranges from 0 to 3, \( \varphi \) is friction angle of joint and \( L_s \) is the shearing length.

This model was extended by Aziz et al. (2016b) to determine the amount of peak shear load when concrete blocks were not in contact with each other.

\[ \tau_p^b = \left( \frac{a_0}{2} + \sum_{n=1}^{3} [a_n \cos \left( n \cos^{-1} \left( \frac{-4a_2 + \sqrt{(16-48a_1 a_3 + 144a_3^2)}}{24a_3} \right)) \right) \right] \tan(\varphi) = \tan(\varphi_b) + c \]  
\hspace{2cm} (2.50)

where, \( \varphi_b \) is the concrete surface basic friction angle which was measured as 26.9°. A comparison between the performance of plain cable bolts, with 5 kN pretension load
using Stratabinder HS grout, with and without friction between concrete block surfaces was conducted. It was observed that the shear strength decreased by 30% when the contact between concrete blocks was eliminated. It means that the shear strength of cable bolts was 70% of the total shear strength of the system. Moreover, the result of the experimental study showed good agreement with analytical simulation.

### 3.6 Summary

Numerous analytical models have been proposed around the world to simulate the axial and shear performance of tendons subjected to pull out test and shear test (single and double shear tests) methodologies.

The main conclusions drawn from this chapter are:

- The shear stresses occur mostly at the interface between the rock bolt and rock mass during the pull test. Thus, it is important that the rock-bolt interface provides enough shear resistance against shear stresses to transfer the loads between the rock bolt and the rock mass.
- Characteristics of the rock bolt, grout and rock mass are important factors in axial performance of rock bolts.
- Cohesion, friction and mechanical interlock are three components in bond strength.
- Researchers believed that inclined rock bolts have higher shear resistance however Hibino and Motojima (1981) stated that there no difference was observed due to the bolt inclination when studying the performance of un-grouted bolts installed in concrete blocks subjected to shearing. Also, it was concluded that pre-tensioning does not affect the shear resistance of bolt although it decreased the shear displacement at peak shear load.
Most of the studies were based on the rock bolt performance and still there is a lack of data in the load transfer capacity of cable bolts used as a secondary support system in coal mines. A number of experimental studies have been conducted on the shear performance of cable bolts but there is no systematic data available on this topic. Thus, the aim of this study is to research the performance of pre-tensioned fully grouted cable bolts subjected to single and double shear tests. This novel laboratory based study coupled with analytical and numerical models aims to determine the performance of cable bolts through the elastic stage up to the failure stage.
Chapter IV

SHEAR BEHAVIOUR OF PRE-TENSIONED FULLY GROUTED CABLE BOLTS SUBJECT TO DOUBLE SHEARING

4.1 Introduction

This chapter reports on the study of the shear behaviour of cable bolts subjected to double shearing. The effect of cable bolt type, surface profile type, cementitious and resin grout type, and pretension load were investigated. Laboratory studies were conducted using concrete blocks with smooth joint surfaces and all tests were carried out in a 500 tonne servocontrolled compression testing machine. Two series of double shear tests, with and without joint surfaces being in contact with each other, were carried out at controlled test conditions of rate of loading and pretension loads.

4.2 Experimental procedure

The experimental procedure consisted of shear testing of cables in concrete blocks under two test conditions of;

a) Shear testing of cables with the sheared joint surfaces of the mortar concrete blocks in contact with each other,

b) Shear testing of cables with the sheared joint surfaces of the concrete blocks not in contact with each other.
4.2.1 Concrete block casting

Three concrete blocks cast for each double shear test are shown in Figure 4-1. The outer 300 mm side cubes were the side blocks for the central rectangular block 450 mm long and 300 x 300 mm in cross section.

![Figure 4-1 Schematic design of concrete blocks](image)

The casting of concrete blocks for the test was carried out in the same steel frame used for the double shear apparatus. Four wooden plates were used to stabilise concrete blocks and separate them from each other during casting. The assembled moulds were held together with an appropriate clamp as indicated in Figure 4-2.
A steel conduit was inserted through three moulds to form a central hole for the cable bolt installation. The diameter of the steel conduit varied depending on the cable diameter. Prior to concrete pour, all wood plates, steel mould sections and the steel conduit were greased using petroleum jelly to prevent fresh concrete from sticking to steel plates, central conduit and timber partitions.

4.2.2 Variations in the printing images of the axial hole in the concrete

Over the course of study, changes were made on how the central cable installation hole was prepared. During the early stage of establishing the central hole in the concrete blocks, appropriate diameter steel tube conduits were used to create central axial hole in the concrete. Once the poured concrete was set, the hole was rifle reamed to the required diameter of the tested tendon. Later on, the process of rifling the hole was replaced in favour of printing rifling image in the hole wall directly by wrapping a 3 mm twin core electrical wire around the steel rod as shown in Figure 4-3, thus enabling printing rifling. This was followed by concrete pour. Also, four 20 mm diameter plastic conduits

Figure 4-2 Prepared moulds for double shear test
were placed vertically along the central line of the mould to print vertical holes over the indented central hole to permit access to the axial hole for cable grout injection purpose. The steel conduit and wrapped electrical wire were then removed after six hours of pouring allowed concrete to semi harden, leaving the print of a rifled holes in the concrete blocks.

![Concrete blocks casting assembly with electrical wire for rifling purpose](image)

**Figure 4-3** Concrete blocks casting assembly with electrical wire for rifling purpose

### 4.2.3 Curing of concrete blocks

Concrete blocks with UCS of 40 MPa were used for various tests. During earlier tests blocks mortar was used, made from sand, cement, in this situation the holes were rifled by reaming. Mortar was soon replaced with concrete, when rifling of the hole was easily printed during concrete pouring stage. The composition of the concrete consisted of sand, cement, 10 mm aggregate and water. In order to achieve 40 MPa strength, The water: cement: sand ratio was 1:2.73:6.64 respectively.

The volume of all three blocks (moulds) for each double shear test was in the order of 0.1 m$^3$ [(300+300+450) mm x 300 mm x 300 mm]. Figure 4.4 shows various stages of concrete blocks preparation.
During concrete pouring stage, 100 mm diameter, 200 mm height concrete cylinder samples were prepared for strength properties of the concrete.

Once set concrete blocks were removed from the steel moulds, were kept in a moist environment for 28 days curing purpose. Vertical holes were initially drilled on top of each block to permit grouting of the installed cable bolt in the double shear assembly. Two holes were drilled on top of the middle block and one hole on each of the corner concrete blocks for grout injection. These vertical holes were also printer on using 20 mm diameter plastic pipes to be removed, leaving vertical holes to be used for grouting purpose as shown in Figure 4-5.
Figure 4- 5 Double shear assembly after casting the concrete

Figure 4-6 shows the concrete samples tested for failure 28 days after casting; the average strength of the concrete was 40 MPa.

Figure 4- 6 UCS test on concrete cylindrical samples

4.2.4 Sample testing procedure

Concrete blocks were kept in moist environment to cure for minimum period of 28 days, prior to testing. In order to gauge the nature of joint surface contacts, each sheared joint surface was painted using special painting pattern shown in Figure 4-7. Each
surface was divided to 36 squares that helped to precisely determine the contact surface area after the test.

![Painting the concrete block to show the crush area](image)

**Figure 4-7** Painting the concrete block to show the crush area

It was essential to calculate the shear strength of the concrete blocks’ joint surfaces due to type of the shearing process. The determination of the concrete face friction value was essential for determining the level of shearing load spent of the rubbing concrete joint surfaces during the shearing process, allowing the true determination of the shearing strength of the tested cable and at the desired value of the applied axial stress.

To achieve this objective, a double shear test was undertaken without installing the cable bolt across the concrete blocks. Concrete blocks were placed back in the steel moulds. Two 30 mm steel plates with dimension of 500 x 300 x 300 mm$^3$ and four 24 mm diameter bolts, 1480 mm in length were used for holding the concrete blocks together. Both ends of each bolt were naturally threaded for the packing and loading purpose. The schematic design by the AutoCAD software is shown in figure 4-8.
The load cell with the strength of 69 MPa (10,000 Psi, 700 Bar and 30 tonne) and the ram with 30 t capacity were used for this test to measure the applied axial load during pre-tensioning and during the test (Figure 4-9).

The shear load was applied to the middle concrete block vertically for the maximum shear displacement of 70 mm. The axial load was fixed as 50 kN for vertical shear displacement between 0 and 20 mm. Then, the axial load was increased to 100 kN for
displacement from 20 mm to 40 mm. In the next stage, the applied axial load to the concrete block was further increased to 200 kN for displacements between 40 mm and 55 mm. The last stage of loading was the 250 kN axial load applied to the middle concrete block for displacement of final 70 mm. All the shear loads were measured and recorded into a data taker and the results are shown in Figure 4-10. The Mohr-Coulomb criterion was used to determine the friction angle between concrete blocks, which was measured as 26.94°.

![Shear load -Displacement](image)

**Figure 4-10** Test results of the concrete blocks sliding test

**4.2.5 Procedure of preparing the double shear assembly by cable bolt installation**

Prior to double shear assembling for testing of each cable, all cables with central grouting holes were fully grouted beforehand, and at least one week prior their installation in the double shear assembly, as these grouting holes were no longer needed for grouting.

More than 12 double shear tests on different types of cable bolt subjected to various pretension loads were conducted. In order to monitor axial loads generated on the tested cable bolt during pre-tensioning and the shearing test, two 60 t load cells were
incorporated. The cable bolt was retained in tension by installation of two sets of barrels and wedge. After pre-tensioning the cable bolt, chemical resin or cementitious grouts were injected in to the holes on the top of concrete blocks depending on the test requirements to encapsulate the cable bolt. As shown in Figure 4-11, tests were conducted seven days after pre-tensioning and encapsulation of the cable bolt. The 500 t machine was used to perform double shear tests. Figure 4-12 shows the installed cabled bolt in the double shear test assembly in the 500 t machine. The rate of shear displacement was set by the digital controller to 1 mm/min. The shear load was applied to the sample using a hydraulic jack located on top of the instrument. The middle concrete block was moved in a vertical direction. The amount of shear and normal load and shear displacement were recorded by the data taker.

Figure 4-11 Sample preparation after pouring resin
4.2.6 Types of cable bolts

Table 4-1 shows properties of different types of cable bolts used in double shear tests.

<table>
<thead>
<tr>
<th>Cable bolt type</th>
<th>Wire Strand No.</th>
<th>Dia. (mm)</th>
<th>Typical Strand Yield Strength (kN)</th>
<th>Typical strand breaking load (t)</th>
<th>Lay length (mm)</th>
<th>Elongation at strand failure (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ID Superstrand</td>
<td>19</td>
<td>21.8</td>
<td>525</td>
<td>60</td>
<td>300</td>
<td>6-7</td>
</tr>
<tr>
<td>Plain Superstrand</td>
<td>19</td>
<td>21.8</td>
<td>525</td>
<td>60</td>
<td>300</td>
<td>6-7</td>
</tr>
<tr>
<td>ID TG</td>
<td>9</td>
<td>28</td>
<td>568</td>
<td>65</td>
<td></td>
<td>5-7</td>
</tr>
<tr>
<td>Plain SUMO</td>
<td>9</td>
<td>28</td>
<td>568</td>
<td>65</td>
<td>400</td>
<td>5-7</td>
</tr>
<tr>
<td>ID SUMO</td>
<td>9</td>
<td>28</td>
<td>568</td>
<td>65</td>
<td>400</td>
<td>5-7</td>
</tr>
<tr>
<td>Garford</td>
<td>7*2</td>
<td>15.2*2</td>
<td>490</td>
<td>53</td>
<td>180</td>
<td>5</td>
</tr>
<tr>
<td>Secura HGC</td>
<td>9</td>
<td>30</td>
<td>565</td>
<td>63</td>
<td>500</td>
<td>4.45</td>
</tr>
<tr>
<td>Plain 19 wire</td>
<td>19</td>
<td>21.8</td>
<td>-</td>
<td>-</td>
<td>300</td>
<td>-</td>
</tr>
<tr>
<td>Plain MW10</td>
<td>10</td>
<td>31</td>
<td>-</td>
<td>70</td>
<td>600</td>
<td>5-6</td>
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<tr>
<td>Spiral MW9</td>
<td>9</td>
<td>31</td>
<td>-</td>
<td>62</td>
<td>600</td>
<td>5-6</td>
</tr>
</tbody>
</table>
4.2.7 Types of used grouts and resin

Mirza et al. (2016) studied the mechanical properties of BU 100 and Stratabinder HS grout. The comparison of the UCS of BU 100 and Stratabinder grouts in 1, 7, 14 and 28 days of curing are shown in Figure 4-13. The UCS of BU 100 was higher than the Stratabinder grout in day 1 however the UCS of Stratabinder was higher than the BU 100 for longer curing periods of 7, 14 and 28 days.

Grout samples were subjected to three cyclic loadings at the loading rate of 1 mm/min for the determination of modulus of Elasticity (E) in compression. The maximum amount of loading in each cycle was limited to 80% of the failure load however the load was increased monotonically after three loading cycles. BU 100 showed a lower E values when compared with Stratabinder (Mirza et al., 2016).
4.3 Double shear test of pre-tensioned fully grouted cable bolts when concrete blocks are in contact (MKII)

The first series of double shear tests were conducted when the concrete blocks were in contact with each other. Therefore, the shear strength of the system is the combination of cable bolt’s shear strength and the friction force between concrete block surfaces.

Table 4-1 shows cable properties, pretension load and the test results for the pre-tensioned fully grouted cable bolts when the concrete blocks are in contact with each other. 40 MPa mortar/concrete blocks were used.

It was observed that Secura HGC cable bolt with 250 kN (25 t) pretension load had the highest peak shear load compared with other types of cable bolt subjected to different pretension loads. However, the indented SUMO cable bolt with pretension load values of 10 and 250 kN (25 t) had the lowest peak shear loads. This may be due to the decrease in the cross sectional area of indented SUMO cable bolts due to indentation, which decreases its strength in shearing. Another important factor in the shear strength of a cable bolt is a lay length. The lay length for Secura HGC cable bolt is 500 mm even though it is 400 mm for SUMO cable bolt. Shorter lay length leads to greater bending load particularly in shorter length of cable bolt thus decreasing its strength.
Table 4-2 Test results for pre-tensioned fully grouted cable bolts when 40 MPa concrete/Mortar blocks are in contact

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Product name</th>
<th>Cable Ø (mm)</th>
<th>Hole dia. (mm)</th>
<th>Bonding agent</th>
<th>Pretension load (t)</th>
<th>Axial load at peak shear load (kN)</th>
<th>Peak shear load (kN)</th>
<th>Net Peak shear load per side at 70% (kN)*</th>
<th>Shear displ. At peak shear load (mm)</th>
<th>Contact coefficient (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>ID Superstrand</td>
<td>21.8</td>
<td>28</td>
<td>Oil based resin</td>
<td>25</td>
<td>400.49</td>
<td>1116</td>
<td>391</td>
<td>74.49</td>
<td>86.1</td>
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<tr>
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<td>21.8</td>
<td>28</td>
<td>Oil based resin</td>
<td>25</td>
<td>429.57</td>
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<td>89</td>
</tr>
<tr>
<td>3</td>
<td>ID TG</td>
<td>28</td>
<td>42</td>
<td>TD80 Grout</td>
<td>25</td>
<td>410.04</td>
<td>1208</td>
<td>422.9</td>
<td>73.85</td>
<td>83.3</td>
</tr>
<tr>
<td>4</td>
<td>ID SUMO</td>
<td>28</td>
<td>42</td>
<td>TD80 Grout</td>
<td>25</td>
<td>307.6</td>
<td>828</td>
<td>289.6</td>
<td>32.76</td>
<td>61</td>
</tr>
<tr>
<td>5</td>
<td>ID SUMO</td>
<td>28</td>
<td>42</td>
<td>TD80 Grout</td>
<td>10</td>
<td>280.28</td>
<td>976</td>
<td>341.6</td>
<td>60.58</td>
<td>81</td>
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<tr>
<td>6</td>
<td>Plain SUMO</td>
<td>28</td>
<td>42</td>
<td>TD80 Grout</td>
<td>25</td>
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<td>1422</td>
<td>504.7</td>
<td>58.76</td>
<td>75</td>
</tr>
<tr>
<td>7</td>
<td>Plain SUMO</td>
<td>28</td>
<td>42</td>
<td>TD80 Grout</td>
<td>10</td>
<td>396.61</td>
<td>1318</td>
<td>461.3</td>
<td>78.99</td>
<td>83.3</td>
</tr>
<tr>
<td>8</td>
<td>Garford twin</td>
<td>15.2x2</td>
<td>55</td>
<td>BU100 Grout</td>
<td>0</td>
<td>339.64</td>
<td>1002</td>
<td>350.7</td>
<td>81.11</td>
<td>69.4</td>
</tr>
<tr>
<td>9</td>
<td>Secura HGC</td>
<td>30</td>
<td>42</td>
<td>FB400</td>
<td>25</td>
<td>461.76</td>
<td>1601.4</td>
<td>560.5</td>
<td>82.21</td>
<td>83.3</td>
</tr>
<tr>
<td>10</td>
<td>Secura HGC</td>
<td>30</td>
<td>42</td>
<td>Carbothix resin</td>
<td>10</td>
<td>487.81</td>
<td>1544</td>
<td>540.4</td>
<td>98.59</td>
<td>88.9</td>
</tr>
<tr>
<td>11</td>
<td>Plain 19 wire</td>
<td>21.7</td>
<td>28</td>
<td>Stratabinder HS</td>
<td>10</td>
<td>634.05</td>
<td>1466.5</td>
<td>513.3</td>
<td>72.74</td>
<td>80.5</td>
</tr>
<tr>
<td>12</td>
<td>Plain 19 wire</td>
<td>21.7</td>
<td>28</td>
<td>Stratabinder HS</td>
<td>0</td>
<td>380.26</td>
<td>1291.3</td>
<td>452.0</td>
<td>85.58</td>
<td>83.3</td>
</tr>
</tbody>
</table>

*Net shear load per side excludes around 30% of the vertical shear load spent on joints surface shearing
Figures 4-14 to 4-26 show test results of the tested cables measured shear and axial loads. The kink in the 5 mm of shear displacement may be due to the adjustment in the barrel and wedge during the early stages of loading. Each figure shows the result of the double shear test, which consists of the shear load measured by compression machine and subsequently recorded in the data taker and the axial loads measured by the load cells. Each shear load profile consists of three stages; elastic, strain softening and failure. The elastic stage is the first stage up to the yield load when the performance of the system is elastic. This means that the deformation in the system due to loading is temporary and reversible. The strain softening stage occurs between yield load and peak load. This stage is a linear stage but it has smaller slope compared with the elastic stage. This is not reversible and continues until the sample reaches the peak load which represents the highest amount of shear load that the sample can carry based on the resistance of cable bolt and friction between concrete blocks. After the peak load, the strand wires of cable bolt starts to break. The shear load drops due to breakage of each wire. When the diameter of the sample is larger the drop would be greater. This continues until all wires snap and the sample fail completely.
**Chapter IV**  
*Shear behaviour of cable bolts subjected to double shearing*

**Figure 4-14** Performance of indented Superstrand cable bolt with 250 kN (25 t) pretension load with contact between concrete block surfaces

**Figure 4-15** Performance of plain Superstrand cable bolt with 250 kN (25 t) pretension load with contact between concrete block surfaces
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Shear behaviour of cable bolts subjected to double shearing

Figure 4-16 Performance of indented TG hollow cable bolt with 250 kN (25 t) pretension load with contact between concrete block surfaces

Figure 4-17 Performance of indented SUMO cable bolt with 250 kN (25 t) pretension load with contact between concrete block surfaces
Figure 4-18 Performance of indented SUMO cable bolt with 100 kN (10 t) pretension load with contact between concrete block surfaces

Figure 4-19 Performance of plain SUMO cable bolt with 250 kN (25 t) pretension load with contact between concrete block surfaces
Figure 4-20 Performance of plain SUMO cable bolt with 100 kN (10 t) pretension load with contact between concrete block surfaces.

Figure 4-21 Performance of Garford twin strand cable bolt with 0 kN (0 t) pretension load with contact between concrete block surfaces.
Figure 4-22 Performance of Secura HGC strand cable bolt with 250 kN (25 t) pretension load with contact between concrete block surfaces

Figure 4-23 Performance of Secura HGC strand cable bolt with 200 kN (20 t) pretension load with contact between concrete block surfaces
Figure 4-24 Performance of Secura HGC strand cable bolt with 100 kN (10 t) pretension load with contact between concrete block surfaces

Figure 4-25 Performance of plain 19 wires, 21.7 mm cable bolt with 100 kN (10 t) pretension load with contact between concrete block surfaces
4.3.1 Effect of cable bolt surface profile type

Figures 4-27, 4-28 and 4-29 show the comparison between peak shear loads of different types of cable bolt with similar pretension load to determine the effect of indentation on their performances. The results from Figures 2-27 and 2-28 show that the Secura HGC cable bolt has the highest peak shear load compared with other types of cable bolt. One possible reason is that the lay length for Secura HGC cable bolt is 500 mm which is longer than the other tested cables stated in Table 4-5. The main reason is due to higher cable diameter in Secura HGC (30 mm) in comparison with other cable bolts. On the other hand, the indented SUMO cable bolt has the lowest peak shear load. The comparison between the plain and indented Superstrand and SUMO cable bolt illustrates that the plain cable bolts have higher peak shear load.
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Figure 4-27 Comparison of peak shear load based on the cable bolt surface profile type for 250 kN (25 t) pretension load

Figure 4-28 Comparison of peak shear load based on the cable bolt surface profile type for 100 kN (10 t) pretension load
4.3.2 Effect of pretension load on shear strength of cable bolt

Figure 4-30 shows the effect of pretension load on the peak shear load. Results show there is no constant pattern, but generally by increasing the pretension load, the peak shear load decreases. However results were not consistent in this series of double shear tests because of:

1. Concrete type, mixture and strength,
2. Pretension load,
3. Lack of confinement, and
4. Types of grout encapsulation which was different for different tests.
All these points make the initial double shear test results inconsistent. This may be attributed to the fact that concrete blocks were not fully confined, leading to early concrete fracture prior to cable failure as shown in Figure 4-31. The fractured blocks will permit increased sheared cable displacement, thus emulating the condition of cable debonding. The excessive bending of the cable section will cause more wires of the strand failing in tension rather than in shear and the failure occurs with increased vertical displacement.

As with regard to increased pretension load, the general expectation is that the cable should fail at lower peak failure at less vertical displacement because of increased cable strand stiffness, but the failure of cable in greater displacement and at increased shear load due to the cable emulating debonding. Hence, a critical review will be conducted in the next series of double shearing (MKIII).
4.3.3 Degree of contact between concrete block surfaces

Figure 4-32 shows variations on contact surfaces between concrete blocks during shearing. It is obvious that the level of contact between concrete block surfaces is less than 100% because of disproportional loading of the central block; the central block will be twisted and shifted to one side and the gap created between concrete blocks decreased the contact.
Figure 4-33 shows variations on concrete surfaces between concrete blocks during shearing. As previously mentioned, each surface was divided into 36 squares. The Contact Coefficient (CC) was calculated as:

\[
CC = \frac{\text{Number of damaged squares}}{\text{Total square numbers (}=36)} \times 100
\]  
(4.1)

The amount of CC for experimental tests of this study are summarised in Table 4-6. This value is mostly between 70% and 90%. The axial stress and shear stress can be calculated using the following equations:

\[
\sigma_n = \frac{N}{CA}
\]  
(4.2a)

\[
\tau = \frac{S}{2 \times CA}
\]  
(4.2b)

where, \(\sigma_n\) is the normal stress, \(N\) is the normal load, \(S\) is the shear load and \(CA\) is cable area. The damage on the concrete blocks was mostly concentrated around cable bolts. It was noted that by increasing pre-tensioning, the contact coefficient increases. The reason is that the load applied to the concrete blocks and cable bolt subjected the sheared concrete joint faces tight, thus increasing the contact forces between concrete blocks. This is evident in tests 5 and 7 and also in tests 7 and 8. For instance, CC values for the indented SUMO cable with pre-tensioning forces of 10 t and 25 t, the CC values were 75% and 81% respectively. The stiffness of the cable is greater compared with the concrete block, and this is the reason why the cable bolt carries more of the total shear load.
Table 4-2 showed the percentage of contact areas between concrete blocks for 12 double shear tests. The minimum contact was 61% and the maximum was 89%.

### 4.3.4 Failure modes of cable bolts

Another important result derived from the tests was the modes of failure in the wires of cable bolts. The failure in the wires of cable bolts were mostly the combination of failure in shear and tension as shown in Figure 4-34. The mode of failure in the wires depends on the cable profile surface and position of wire in the cable bolt. It is important to consider the cable area which is being subjected to shear and tensile failure. Table 4-3 shows the result of wires failure in a cable bolts in shear and tension. The test numbers in Table 4-3 are corresponds to Table 4-2. Failure in tension alone is demonstrated by the cone and cup failure while the combined tension/shear failure is depicted and an angle shearing failure. The location of the wire on the cable cross-section during shearing dictates which type of the failure of the strand wire underwent. The fully shear failure of cable bolts is only possible in guillotine style of failures near similar to British Standard tests (metal to metal contact) and also a new method
proposed by McTyer and Evans (2017) as shown in Figure 4-35, however this new method underestimate the shear strength of cable bolts and thus is insufficient. Not the pinching or squeezing effect of metal to metal shearing

![Figure 4- 34 Failure of cable strands in shear and tension](image)

**Figure 4- 34** Failure of cable strands in shear and tension

![Figure 4- 35 Direct shear apparatus proposed and sheared cable by McTyer and Evans (2017)](image)

**Figure 4- 35** Direct shear apparatus proposed and sheared cable by McTyer and Evans (2017)
Table 4-3 Area of cable wires failure in shear and tension

<table>
<thead>
<tr>
<th>Test No.</th>
<th>No. of wires failed in tension</th>
<th>No. of wires failed in shear</th>
<th>No. of wires failed in tension and shear</th>
<th>Tensile failure area (mm$^2$)</th>
<th>Shear failure area (mm$^2$)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6</td>
<td>13</td>
<td>0</td>
<td>81.83</td>
<td>212.58</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>14</td>
<td>5</td>
<td>0</td>
<td>270.24</td>
<td>35.33</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>346.18</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>9</td>
<td>0</td>
<td>0</td>
<td>346.19</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>6</td>
<td>3</td>
<td>0</td>
<td>192.33</td>
<td>153.86</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>8</td>
<td>0</td>
<td>1</td>
<td>307.72</td>
<td>38.46</td>
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</table>
### Table 4-3 Continued

<table>
<thead>
<tr>
<th>Test No.</th>
<th>No. of wires failed in tension</th>
<th>No. of wires failed in shear</th>
<th>Tensile failure area (mm²)</th>
<th>Shear failure area (mm²)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>9</td>
<td>0</td>
<td>346.18</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>9</td>
<td>5</td>
<td>183.86</td>
<td>102.14</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>7</td>
<td>2</td>
<td>269.26</td>
<td>76.93</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>9</td>
<td>0</td>
<td>346.19</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>10</td>
<td>9</td>
<td>191.32</td>
<td>122.27</td>
<td></td>
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<tr>
<td>12</td>
<td>9</td>
<td>8</td>
<td>191.32</td>
<td>104.33</td>
<td></td>
</tr>
</tbody>
</table>
4.3.5 Cable bolt deformation

For most frequently used bolts, the cable bolt deformation print in concrete was typically between 3 and 4 times the bolt diameter. Figures 4-36 and 4-37 show the deformation zone in the concrete block from hinge point to the central face. Then, forming a semi-conical deformation and crushing of surrounding grout and in the concrete blocks. This was measured for all the conducted tests and will the result being between 60 and 80 mm in depth, equal to 3 and 4 times of cable bolt’s diameter.

Figure 4-36 Concrete block crushing due to cable bolt deflection

Figure 4-37 Location of cable bolt deformation
More than 12 double shear tests on different types of cable bolt subjected to various pretension loads were studied. The total shear load carried by the system is the combination of shear force due to shearing of the cable and overcoming the friction force of the joint faces of concrete blocks. The majority of the system’s shear strength was generated by the cable bolt. No debonding was observed in the grout because the cable bolt ends were retained by barrel and wedge and also the extremity side blocks were not of sufficient length to depict cable debonding. The loads were monitored by load cells. If the ends of the cable were not fitted with barrel and wedge, the cable ends would have pulled through and decoupling would occur. Thus, the shear load of the system is the load due to contact faces rubbing friction and the cable bolt. The shear strength among concrete blocks was provided by the friction along the surfaces. As the shear displacement increases, the shear strength of the concrete block decreases because of the damage occurring in the concrete surfaces. The joint friction angle is also an important factor which was measured based on the sliding of the concrete blocks without installation of cable bolts as 27°. When the joint friction angle is higher, it means that the joint has higher strength to resist against shearing. The shear strength of the cable bolt is the total shear strength of broken wires in shear or in tension. This will be discussed in more details in the mathematical analysis in Chapter V. The angle between the cable bolt and the joint is another important factor that required addressing and in this study this angle was kept constant at 90° so that the effect of other parameters including cable profile type, cable type and pretension load were studied. This will be discussed in more details in the mathematical analysis part.
4.4 Double shear of pre-tensioned fully grouted cable bolt when concrete blocks are not in contact (MKIII)

In order to determine the shear resistance of concrete blocks and cable bolts separately, a new series of double shear test was conducted by eliminating the contact between concrete block surfaces to omit friction. The aim of this study is to determine the net shear strength of pre-tensioned fully grouted cable bolt subjected to double shearing.

4.4.1 MKIII Equipment description

A modified double shear apparatus (MKIII) was used to remove the friction between concrete block joint surfaces during the shearing process and to determine pure shear strength of cable bolts. Two open box steel channel braces (U brace), mounted axially on each side of the double shear apparatus assembly and connected to two end plates with dimension of 500 x 340 x 30 mm was employed to prevent the concrete joint faces of the concrete blocks coming in contact with each other as shown in Figure 4-36. During shearing the load was transferred from the U sections to the end plates thus avoiding the friction between shearing blocks. Therefore, the size and thickness of end plates was a critical aspect to carry the axial load. The maximum axial load generated in the system due to the pretension load and during the test did not exceed 600 kN. Therefore, each U section should have 150 kN strength however the end plates had to be strong enough for a 300 kN load. The maximum deflection in the end plate centre was calculated from the following equation:

$$\delta_{\text{max}} = \frac{P l^3}{48 E I}$$  \hspace{1cm} (4.3)

where, $\delta_{\text{max}}$ is the maximum deflection, $P$ is the load applied to the end plate, $l$ is the end plate length, $E$ is elastic modulus and $I$ is the second moment of Inertia.
For the current test, \( l \) is 0.52 m, \( P \) is \( 3 \times 10^5 \) N and \( E \) is \( 2 \times 10^{11} \) Pa. Therefore, for different thickness of the plate, the maximum deflection (\( \delta \)) is shown in Table 4-4. \( b \) is the width of the plate and \( I \) is the second moment of area. Figure 4-38 shows the schematic design of the new double shear instrument which provides a gap between concrete blocks to omit friction. Dimensions of the design are summaries in Table 4-5. Additional support in the form of 250 mm x 250 mm 25 mm plates was added to the end plates to minimise possible plate deflection (less than 1 mm).

**Table 4-4** Maximum deflection in the end plate with different thicknesses

<table>
<thead>
<tr>
<th>Thickness (m)</th>
<th>b (m)</th>
<th>I (m⁴)</th>
<th>( \delta ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.03</td>
<td>0.32</td>
<td>7.2E-07</td>
<td>6.1</td>
</tr>
<tr>
<td>0.04</td>
<td>0.32</td>
<td>1.71E-06</td>
<td>2.57</td>
</tr>
<tr>
<td>0.05</td>
<td>0.32</td>
<td>3.33E-06</td>
<td>1.32</td>
</tr>
</tbody>
</table>

**Figure 4-38** Double shear assembly without friction between concrete blocks
Table 4-5 New double shear test assembly dimension size of plates

<table>
<thead>
<tr>
<th>Plate’s location</th>
<th>Dimension (mm³)</th>
<th>Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>340x300x20</td>
<td>2</td>
</tr>
<tr>
<td>Top</td>
<td>340x450x20</td>
<td>1</td>
</tr>
<tr>
<td>Side</td>
<td>300x300x20</td>
<td>4</td>
</tr>
<tr>
<td>Side</td>
<td>450x300x20</td>
<td>2</td>
</tr>
<tr>
<td>End</td>
<td>520x350x30</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>250x250x25</td>
<td>2</td>
</tr>
<tr>
<td>U section (Open channel beam)</td>
<td>5000 mm</td>
<td></td>
</tr>
</tbody>
</table>

The aim of this study was to determine the effect of cable surface profile and pretension load. Sample preparation and concrete blocks casting were similar to the double shear testing with contact between concrete blocks. Refer to section 4.2 for sample preparation. Figure 4-39 shows the new double shear assembly in 500 tonne machine.

Figure 4-39 Double shear assembly without contact (MKIII)
Table 4-6 shows the result of the conducted tests when there was no contact between concrete blocks. Plain MW10 and Spiral MW9 cable bolts with 0 pretension load did not reach their peak shear loads in 105 mm of shear displacement although the shear strength of Spiral MW9 in 105 mm displacement was 939 kN which was still the highest shear load among different types of tested cable bolts subjected to different pretension loads.
### Chapter IV

Shear behaviour of cable bolts subjected to double shearing

Table 4-6 Test plan for the pre-tensioned fully grouted cable bolts without friction between concrete blocks

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Cable bolt type</th>
<th>Dia. of cable bolt (mm)</th>
<th>Cable geometry</th>
<th>hole dia. (mm)</th>
<th>Pretension (kN)</th>
<th>Peak axial load (kN)</th>
<th>Peak shear load (kN)</th>
<th>Peak shear load per side (kN)</th>
<th>Shear displ. at peak shear load (mm)</th>
<th>Shear strength/UTS (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Plain 19 wire</td>
<td>21.7</td>
<td>Un-bulbed</td>
<td>28</td>
<td>0</td>
<td>400.2</td>
<td>761</td>
<td>380.5</td>
<td>98.8</td>
<td>64.5</td>
</tr>
<tr>
<td>2</td>
<td>Plain 19 wire</td>
<td>21.7</td>
<td>Un-bulbed</td>
<td>28</td>
<td>100</td>
<td>443.9</td>
<td>738</td>
<td>369.0</td>
<td>92.3</td>
<td>62.5</td>
</tr>
<tr>
<td>3</td>
<td>Plain 19 wire</td>
<td>21.7</td>
<td>Un-bulbed</td>
<td>28</td>
<td>0</td>
<td>433.8</td>
<td>884</td>
<td>442.0</td>
<td>76.8</td>
<td>74.98</td>
</tr>
<tr>
<td>4</td>
<td>Plain 19 wire</td>
<td>21.7</td>
<td>Un-bulbed</td>
<td>28</td>
<td>160</td>
<td>403.4</td>
<td>774</td>
<td>387</td>
<td>86</td>
<td>65.6</td>
</tr>
<tr>
<td>5</td>
<td>Plain SUMO</td>
<td>28</td>
<td>Bulbed</td>
<td>42</td>
<td>0</td>
<td>432</td>
<td>886*</td>
<td>443</td>
<td>100</td>
<td>69.8</td>
</tr>
<tr>
<td>6</td>
<td>Plain SUMO</td>
<td>28</td>
<td>Bulbed</td>
<td>42</td>
<td>150</td>
<td>433</td>
<td>852</td>
<td>427</td>
<td>88.2</td>
<td>67.1</td>
</tr>
<tr>
<td>7</td>
<td>ID SUMO</td>
<td>28</td>
<td>Bulbed</td>
<td>42</td>
<td>0</td>
<td>364</td>
<td>814.93</td>
<td>407.5</td>
<td>93.4</td>
<td>64.2</td>
</tr>
<tr>
<td>8</td>
<td>ID SUMO</td>
<td>28</td>
<td>Bulbed</td>
<td>42</td>
<td>150</td>
<td>375.7</td>
<td>766.95</td>
<td>383.5</td>
<td>85.7</td>
<td>60.4</td>
</tr>
<tr>
<td>9</td>
<td>Plain MW10</td>
<td>31</td>
<td>Un-bulbed</td>
<td>42</td>
<td>0</td>
<td>470.36</td>
<td>878.2*</td>
<td>439.1</td>
<td>105</td>
<td>63.9</td>
</tr>
<tr>
<td>10</td>
<td>Plain MW10</td>
<td>31</td>
<td>Un-bulbed</td>
<td>42</td>
<td>150</td>
<td>505.5</td>
<td>923.14</td>
<td>461.6</td>
<td>88.5</td>
<td>67.2</td>
</tr>
<tr>
<td>11</td>
<td>Spiral MW9</td>
<td>31</td>
<td>Un-bulbed</td>
<td>42</td>
<td>0</td>
<td>387.1</td>
<td>939*</td>
<td>469.5</td>
<td>105</td>
<td>77.2</td>
</tr>
<tr>
<td>12</td>
<td>Spiral MW9</td>
<td>31</td>
<td>Un-bulbed</td>
<td>42</td>
<td>75</td>
<td>440.1</td>
<td>907</td>
<td>453.5</td>
<td>89.7</td>
<td>74.6</td>
</tr>
<tr>
<td>13</td>
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<td>Un-bulbed</td>
<td>42</td>
<td>150</td>
<td>397.1</td>
<td>837</td>
<td>418.5</td>
<td>88.47</td>
<td>68.8</td>
</tr>
</tbody>
</table>

* It is not peak shear load because the cable did not break. ** Plain 19 wire, 21.7 mm diameter.
Figures 4-40 to 4-52 show the result of double shear test on different types of cable bolt subjected to different pretension load. Each figure shows the shear load and axial loads from both loads cells versus shear displacement.

**Figure 4-40** Performance of plain 19 wire, 21.7 mm cable bolts with 0 kN pretension load without contact between concrete block surfaces

**Figure 4-41** Performance of plain 19 wire, 21.7 mm cable bolts with 100 kN pretension load without contact between concrete block surfaces
Figure 4-42 Performance of plain 19 wire, 21.7 mm cable bolts with 0 kN pretension load without contact between concrete block surfaces

Figure 4-43 Performance of plain 19 wire, 21.7 mm cable bolts with 160 kN pretension load without contact between concrete block surfaces
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**Figure 4-44** Performance of plain SUMO cable bolts with 0 kN pretension load without contact between concrete block surfaces

**Figure 4-45** Performance of plain SUMO cable bolts with 150 kN pretension load without contact between concrete block surfaces
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Figure 4-46 Performance of indented SUMO cable bolts with 0 kN pretension load without contact between concrete block surfaces

Figure 4-47 Performance of ID-SUMO cable bolts with 150 kN pretension load without contact between concrete block surfaces
**Figure 4-48** Performance of plain MW10 cable bolts with 0 kN pretension load without contact between concrete block surfaces

**Figure 4-49** Performance of plain MW10 cable bolts with 150 kN pretension load without contact between concrete block surfaces
Figure 4-50 Performance of spiral MW9 cable bolts with 0 kN pretension load without contact between concrete block surfaces

Figure 4-51 Performance of spiral MW9 cable bolts with 75 kN pretension load without contact between concrete block surfaces
4.4.2 Effects of pretension load

Figure 4-53 shows the effect of pretension load on the peak shear load of various cable bolts. It was observed that the peak shear load decreases by increasing the pretension load of all the five types of tested cable bolts. The exception to the trend was the peak shear load for the plain MW10 as the cable bolt did not reach its peak shear load at 0 t pretension load. Moreover, the shear displacement at peak shear load decreased by increasing the pretension load. Further tests are planned for MW10 cable bolt. Also note that the four plain 19 wire cable bolt results are arranged in two groups (tests 1 and 4 in one group and tests 2 and 3 in another) because different techniques were used to stop grout leaking out of the blocks during the cable bolt installation process. The reduction on peak shear load with respect to increased pretension is more realistic that the increased values reported when tests were made with joint face contacts.
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Figure 4-53 Effect of pretension load on peak shear load. Note; Plain MW10 with 0 t pretension did not fail.

4.4.3 Effects of cable type

Figures 4-54 and 4-55 show the comparison between peak shear loads of different types of cable bolts subjected to 0 and 15 t pretension loads. The result shows that the plain MW10 had the highest peak shear load at 15 t pretension loads. The comparison in 0 t pretension load was difficult with other tested cables because the plain SUMO, spiral MW9 and plain MW10 did not fail in 100 mm of shear displacement. The higher shear strength of plain MW10 was due to the cable strand having ten wires instead of nine wires of SUMO and MW9 cable bolts. Other features of MW10 cable include high tensile strength of 70 t as shown in Table 4-1, greater wire lay length of 600 mm and plain cable surface profile type.
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**Figure 4-54** Peak shear loads of different types of cable bolts subjected to 0 kN (0 t) pretension load

**Figure 4-55** Peak shear loads of different types of cable bolts subjected to 150 kN (15 t) pretension load
4.4.4 Cable bolt’s mode of failure

Test results show that the failure of cable bolts was mostly a combination of shear and tensile failure which depended on the type of cable, strand diameter and strand position in the cable bolt. Table 4-7 summarises the results of cable bolts’ failure in double shear tests. It was seen that there were two tests that the failure occurs only in tension and only one test that failure occurred in shear. Moreover, it was obvious that the failure in the plain cable bolt was mostly in tension however the failure in the spiral cable bolts mostly occurred in shear. This shows the effect of cable type on the failure mode.

4.4.5 Cable bolt’s deflection

Figure 4-56 shows the deflection in the cable bolt. Result from Table 4-7 illustrated that the deflection in the tests were between 65 mm and 110 mm with the average of 85 mm. The amount of cable deflection is about three times its diameter.

Figure 4- 56 Cable bolt deflection
Table 4-7 Broken area of cable wires in shear and tension and cable deflection when concrete blocks are not in contact

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Cable</th>
<th>Pretension (kN)</th>
<th>Dia. (mm)</th>
<th>No. of wires failed in tension</th>
<th>No. of wires failed in shear</th>
<th>No. of wires failed in tension and shear</th>
<th>Tensile failure area (mm²)</th>
<th>Shear failure area (mm²)</th>
<th>Cable bolt def. (mm)</th>
<th>Diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Plain 19 wire</td>
<td>0</td>
<td>21.7</td>
<td>2</td>
<td>1</td>
<td>0</td>
<td>184.22</td>
<td>129.3</td>
<td>70</td>
<td>3.21</td>
</tr>
<tr>
<td>2</td>
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<td>21.7</td>
<td>6</td>
<td>2</td>
<td>0</td>
<td>91.68</td>
<td>30.60</td>
<td>60</td>
<td>2.75</td>
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<td>21.7</td>
<td>15</td>
<td>0</td>
<td>0</td>
<td>253.85</td>
<td>0</td>
<td>65</td>
<td>2.98</td>
</tr>
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<td>4</td>
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<td>9</td>
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<td>113.83</td>
<td>85</td>
<td>3.90</td>
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<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>90</td>
<td>3.21</td>
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<td>110</td>
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<td>76.97</td>
<td>95</td>
<td>3.39</td>
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<tr>
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<td>N.A.</td>
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<td>N.A.</td>
<td>100</td>
<td>3.23</td>
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<tr>
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<td>2</td>
<td>2</td>
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<td>105</td>
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<td>N.A.</td>
<td>N.A.</td>
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<td>N.A.</td>
<td>90</td>
<td>2.90</td>
</tr>
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<td>3</td>
<td>287.02</td>
<td>154.55</td>
<td>105</td>
<td>3.39</td>
</tr>
</tbody>
</table>
4.5 Comparison between performance of cable bolts with and without contact between concrete block surfaces

Figures 4-57 and 4-58 show the comparison between the result of plain 19 wire, 21.8 mm cable bolt with 0 and 10 t pretension load subjected to double shear tests with and without contact between concrete blocks surfaces.

The comparison between the performance of the plain 19 wire, 21.7 mm cable bolt without pretension load shows that the peak shear loads were 1291 kN and 761 kN, when the concrete blocks were with and without contacts. The shear load is the combination of the shear load provided by the breakage of the cable bolt and the sliding between the concrete blocks when the concrete blocks are in contact with each other.

When concrete blocks are not in contact, the peak shear load decreases because it only includes the load due to the failure of the cable bolt. The performance of the plain 19 wire cable bolt with 10 t pretension load are 1369 and 738 kN when the concrete blocks are with and without contact respectively. When the concrete blocks are not in contact, the peak shear load is 54% of the peak shear load when the concrete blocks are in contact with each other. This shows a great contribution of concrete blocks in the total shear strength of system. Moreover, the comparison between the shear displacements at peak shear load shows that the shear displacement at peak shear load was higher when there is no contact between concrete blocks. This is due to higher freedom in movement of concrete blocks.
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Figure 4-57 Comparison between performance of plain 19 wire, 21.7mm cable bolt with 0 t pretension load when concrete blocks are in use and are not in contact with each other

Figure 4-58 Comparison between performance of plain 19 wire, 21.7 mm cable bolts with 10 t pretension load when concrete blocks are in use and are not in contact with each other
The peak shear load is low in MKIII as opposite to what is happening with MKII. This is because the shear load in MKII is a combination of cable bolt shear strength and friction between concrete blocks however the shear load in MKIII only includes the cable bolt strength. It is clear from the results shown in Table 4-7 that because of the most wires of the tested cable are failing in tension, the final shear failure load appears to be closer to failure in tension as demonstrated by the axial load build up per side being nearly equal to shear failure load as shown in Figure 4-59.

\[ \text{Figure 4-59 Comparison between shear loads and axial loads in MKIII} \]
4.6 Summary

The performance of different types of cable bolts subjected to different pretension loads were studied using a double shear assembly with and without contact between concrete block surfaces. The following conclusions were drawn:

- The shear behaviour of each cable bolt has three behaviour stages of elastic, strain softening and failure.
- Cable type, pretension load, failure angle, lay length and the concrete block strength are important factors in the peak shear load of pre-tensioned fully grouted cable bolts.
- On average, the contact between concrete block surfaces was 80% of the total surface area. However, the pure rubbing of the joint faces may be misleading as a significant damage can occur on joint surface during the snapping phase of the cable as one side of the central block leans against outer extremity block.
- The plastic hinge location mostly occurs at 60 to 80 mm from the contact surfaces when the concrete blocks were in contact with each other; however the hinge point location was found to start at deeper location up to 110 mm when the shearing blocks are not in contact with each other.
- The peak shear load occurs in smaller shear displacement by increasing the pretension load.
- The peak shear load for the plain cable is higher than for the spiral/indented one at a similar amount of pretension load because the cross sectional area for plain cable is more than for the spiral cable bolts.
- The ultimate tensile strength, lay length, number of wires, surface profile type (plain, spiral and indented) are important factors in the performance of cable bolts in...
shear. The Plain MW10 has the highest amount of peak shear load due to its highest UTS, 600 mm lay length and plain surface profile type.

- No debonding occurred due to the use of barrel and wedge in the double shear assembly.

- Increased pretension in double shearing without proper confinement leads to greater shear failure load and greater displacement. In this situation, the deflection is greater causing failure in tension instead of shear. In other words, the failure due to tensile is greater that in shear. Lack of concrete effective confinement is an influencing factor. Basically the cable behaves less stiff with increased pretension load. The shear load when testing in double shear without face contacts, the final shear failure load appeared to be closer to failure in tension rather than in shear as the axial load build up per side being nearly equal to shear failure load.
Chapter V

SHEAR BEHAVIOUR OF PRE-TENSIONED FULLY GROUTED CABLE BOLTS SUBJECTED TO SINGLE SHEARING

5.1 Introduction

Laboratory studies are essential to determine similarities and differences between the shear performance of pre-tensioned fully grouted cable bolts subjected to double and single shear testing. The main objective of this study is to investigate the shear behaviour of fully grouted cable bolts under newly developed single shear apparatus. Thus, a laboratory investigation was conducted based on a comprehensive experimental program using different types of cable bolts and various pretension loads.

5.2 Laboratory investigation

The laboratory investigation involved single shear testing of cable bolts using a newly developed and integrated single shear apparatus. Topics discussed include the description of the integrated Megabolt single shear testing apparatus, casting of concrete cylinders, encapsulation of the cables in concrete cylinders, testing procedure and analysis of the test results.

5.2.1 Single shear apparatus

On loan from Megabolt Australia, the new single shear apparatus was used for testing of cable bolts. This horizontally aligned integrated system consisted of a shearing rig and a 120 t compression machine. The shearing cylinder consisted of two sections, each containing 1.8 m concrete cylinders. Concrete cylinders were covered by steel clamps, provided confinement during shearing. The shear displacement was applied by four hydraulic rams located beneath the apparatus where the applied shear load was measured by the calibrated pressure transducer and analogue gauge (Figure 5-1). Either
a hand pump or a power pack of a suitable capacity applied the hydraulic pressure for compression testing machine legs. The pressure in the manifold was monitored with a pressure transducer in conjunction with an analogue pressure gauge. The rate of loading was applied manually which was not constant however the aim was to apply a constant load at the rate of around 1 mm/min (0.018 mm/sec), in line with BS7861-2 standards. The displacement at the shearing plane was measured using a Linear Variable Differential Transformer (LVDT) as shown in Figure 5-2. Two other LVDTs were mounted on the cable ends enabled monitoring cable debonding. The instrument was associated with an accurate data taker that collected data at the desired time interval. A data taker was used to collect data during the test.

![Figure 5-1 Single shear testing apparatus](image-url)
5.2.2 Sample preparation

Formatube cylindrical cardboards, 250 mm in diameter and 900 mm in height were used for casting concrete cylinders. A steel rod with 30 mm diameter and length of 1000 mm was installed in the centre of the cylinder to create the borehole. An 8 mm diameter PVC tube was wrapped around the steel rod for rifling the hole during casting (Figure 5-3).
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Figure 5-3 Concrete moulds configuration

The concrete mix was prepared by an external body, Baine Concreting Services, with 10 mm aggregates and a UCS of 40 MPa was poured in cylindrical moulds. A slump test was conducted before pouring the fresh concrete to ensure the required moisture and consistency. The slump of 80 mm was resulted in fresh concrete to assure integrity in moisture and uniformity of the mix to maintain cast concrete consistency and strength. Subsequently, the fresh concrete was poured into the moulds and vibrated to remove any entrapped air bubbles from the poured concrete. The top surface was smoothened as shown in Figure 5-4. 32 cylinder blocks were cast in each set of casting. Once the concrete blocks cured, they were taken out from the Formatube cardboard moulds. To retrieve concrete blocks from the moulds, the wooden structures holding the concrete filled moulds were dismantled. Then, the plastic wire (clear PVC tube) was pulled out and the steel rod located in the centre of the concrete block was tapped out. The
Formatube cardboard was cut and split axially and the concrete blocks were taken out and kept to cure for at least 28 days to reach the desired 40 MPa UCS strength. The strength of the concrete was confirmed from USC tests on 200 mm long and 100 mm diameter concrete cylinders prepared during concrete pour.

![Prepared concrete cylinders](image)

**Figure 5-4** Prepared concrete cylinders

### 5.2.3 Butt gluing of concrete cylinder ends

Every two 900 mm cylinders were glue butted to each other, forming a 1.8 m long unit. For this purpose, the first half of the steel clamp was positioned on the “Base frame” and covered with a plastic sheet. Then, one end of the 900 mm concrete block was placed in the centre of the clamp as shown in Figure 5-5.
Epirez non-sag epoxy binder and hardener mix with a ratio of 3:1 was used for gluing the butting end surface of the concrete blocks. The epoxy was used to smear the concrete cylinder surfaces but it was not applied in the vicinity of the centre hole, keeping the centre hole clear for cable encapsulation as shown in Figure 5-6.

To attach concrete blocks and glue them effectively, a bolt was positioned in the hole and tightened to attach joints completely after smearing joint surfaces with epoxy. The
whole assembly was left undisturbed for a period of 30 minutes. Subsequently, the bolt was removed and the other top half of the outer confining steel clamp was placed and tightened (Figure 5-7). Each clamp was capable of carrying 30-40 t. For each test, four concrete blocks were required to provide a total length of 3.6 m.

Two sets of concrete blocks were attached together in the pre-tensioning and grouting frames. The primary clamp was used for the middle joint reinforcement during pre-tensioning and grouting. The other parts of concrete blocks were covered with the secondary clamps to provide full confinement during shearing.

The Sika Bond Spray Fix with high strength contact adhesive was used to cover the grout adaptor plate. Then, the adaptor plate was positioned and the internal surface was sprayed (Figure 5-8). Finally, the grout adaptor plate and concrete block surface were attached together as shown in Figure 5-9.
The primary clamp was placed in the centre of the pre-tensioning and grouting frames. The primary clamp was used to protect the integrity of the joint and allow for ease of lifting with an eye bolt on top when transporting the assembled sample to the single shear rig. The concrete block surface and the neoprene seal were sprayed at the centre of the concrete blocks. The neoprene seal was attached to the concrete block surface of each side. Then, the other side of neoprene seal and the Teflon sheet were sprayed and attached together. Each surface required to be kept undisturbed for two minutes after spraying (Figure 5-10). The intention of using neoprene seal was to seal the concrete.
block for grouting. Each Teflon sheet had a thickness of 2 mm, providing a 4 mm gap between concrete blocks in shear joints. The Teflon sheet ring had an outer diameter of 245 mm and the inner diameter of 85 mm. The neoprene seal had the external diameter of 85 mm, allowing the Teflon sheet to position around it precisely. The internal diameter of the neoprene seal was set to 45 mm, facilitating the cable bolt installation. Thereafter, the same procedure was carried out to make a 3.6 m concrete cylinder for testing. The other half of the primary clamp was positioned on top of the other half as shown in Figure 5-11.

Figure 5-10 Attaching Teflon sheet and neoprene seal to the middle joint

Figure 5-11 Application of primary clamp to the middle joint
5.2.4 Cable bolt installation

Initially, it was decided to monitor the strain on cable wires using strain gauges. Strain gauges were installed on one clean strand wire of the cable bolts to monitor debonding in the cable bolt during shearing. A small areas of one strand wire was sanded and cleaned with an alcohol based cleaner to ensure impurities on the surface were removed. Araldite epoxy resin and hardener at a ratio of 1:1 was applied to the surfaces where strain gauges were placed on the wire surface and taped in place with adhesive tape. Once the strain gauges glue set, the tape was removed as shown in Figure 5-12. In order to connect the gauge to the data logger during testing, low resistance electrical wires were soldered onto the strain gauge conductors. Silastic glue was used to spot glue and anchor the connecting wires of the strain gauges along the cable surface to ensure their integrity during cable insertion and subsequent grouting process. It was essential to minimise the effect of silicon glue in contact with the cable wire surface, so that its presence will not influence cable debonding. Figure 5-13 shows the location of strain gauges installation on the cable bolts. In subsequent tests and following testing of plain MW10 and spiral MW9 cables, the strain gauges were replaced with LVDTs. The reasons for replacing strain gauges with LVDTs will be discussed later in this chapter.

Figure 5-12 Application of strain gauge on cable bolt’s surface
Once the strain gauges and lead wires were mounted on the cable, each instrumented cable bolt was then inserted carefully in the concrete cylinder hole. To avoid sliding during pretensioning, the cable bolt was fixed in place by using a jack plate, spacer, nut, 62.5 mm jack and the barrel and wedge. Then, the cable bolt was pre-tensioned at the opposite side designated for grouting due to the required elements to avoid sliding of the cable bolt during pre-tensioning (Figure 5-14). The instrument was designed in such a way that prevents subjection of pre-tension load to the concrete cylinders (Figure 5-15). All cables were not subjected to pretension when shear tested.
5.2.5 Grouting the cable bolt

The pre-tensioning and grouting frame was tilted at an angle of 65° to the horizon for the bottom up grouting of the cable (Figure 5-16). There were two main reasons for this:

- To replicate field conditions; the cable is usually installed at an angle, and
To remove any air bubbles remaining inside the annulus area and hence to ensure full encapsulation.

Stratabinder HS grout was used for the cable bolt encapsulation with the w: c ratio of 7.5 to 20 kg. One bag of cement (20 kg) was used for each set of cable bolt encapsulation. The Stratabinder grout provides 72 MPa of UCS after 28 days of curing.

To obtain a smooth mixture without any lumps, the cement was added uniformly to water. The tilted concrete blocks after grouting was kept undisturbed for 28 days for grout curing.

![Figure 5-16 Locating concrete cylinders in angle for grouting purpose](image)

5.2.6 Testing procedure

Once each encapsulated sample grout was cured for 28 days, it was disassembled from the tension frame and carefully mounted on the integrated shear testing rig, ready for testing. Steel clamps were placed around the concrete blocks to provide a confinement to the sample, apart from the primary clamp adjacent to the shearing plane. The aim of using confinement on the medium was to provide a more accurate replication of in situ conditions of the forces applied to the cable bolt from adjacent strata.

During early stage of cable strain measurement approach, strain gauges and dial gauges were used to monitor any possible debonding of the cable, however LVDTs replaced
the use of strain gauges in later tests. LVDTS were set up at each end of the cable extremity after removing the nut, spacer, grouting trumpets and barrel and wedge from the sample. A hydraulic power pack was connected the hydraulic rams and to commence the testing, the pressure was applied manually which was tried to keep constant although the shear rate was not constant for different tests.

5.3 Experimental results

Table 5-1 lists the number of tests carried out in this programme of testing. The table provides details of eight different tested cables and their characteristics. The cables were tested in 40 MPa strength concrete blocks, being encapsulated using Stratabinder grout. The table also include an additional three tests (17, 18 and 19) carried out on plain MW10 cables fitted with LVDTS for monitoring debonding. The repetition of the plain MW10 cables was undertaken because there were some concerns about the effect of silastic on early debonding. Silastic glue was used to glue the strain gauge wires along the cable strand wires.

The results of the single shear tests for different cable bolts subjected to different pretension load are plotted in Figure 5-17 through 5-25. Figure 5-17 shows the result of the plain MW10 cable with strain gauges (tests 1, 2 and 3) and Figure 5-25 shows the load-displacement of the plain MW10 in the second round of tests using LVDTs ( test 17, 18 and 19). It is obviously clear that all plain MW10 cables were debonded in both test series. Other debonded cables were plain SUMO and plain Superstrand (21.7 mm 19 wire) cable bolt. All other tested cables, which were mostly of indented type, did not fail. Interestingly, Secura HGC cable with 40% of indented outer wires did not debond either.
### Table 5-1 Summary of single shear test results

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Product Name</th>
<th>Cable diameter (mm)</th>
<th>Capacity (t)</th>
<th>Cable geometry</th>
<th>Pretension load (t)</th>
<th>Peak Shear load (t)</th>
<th>Shear displ at peak shear load</th>
<th>Cable debonding</th>
<th>Shear displ corresponding to debonding</th>
<th>Peak shear load/UTS (%)</th>
<th>Rate of loading (mm/min)</th>
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<tbody>
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<td>63</td>
<td>6 bulbs</td>
<td>0</td>
<td>40.43</td>
<td>40.91</td>
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<td>ID- SUMO</td>
<td>28</td>
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<td>6 bulbs</td>
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<td>37.36</td>
<td>30.9</td>
<td>No</td>
<td>0</td>
<td>59.4</td>
<td>13</td>
</tr>
<tr>
<td>13</td>
<td>ID-TG</td>
<td>28</td>
<td>60</td>
<td>Un-bulbed</td>
<td>0</td>
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<td>51.3</td>
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<td>ID-TG</td>
<td>28</td>
<td>60</td>
<td>Un-bulbed</td>
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<td>Superstrand-P</td>
<td>21.7</td>
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<td>52.40</td>
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<td>Yes</td>
<td>42</td>
<td>85.7</td>
<td>2.5</td>
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<tr>
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<td>Garford-P</td>
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<td>2*27</td>
<td>Bulbed</td>
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<td>MW 10-P*</td>
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<td>70</td>
<td>Un-bulbed</td>
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<td>80</td>
<td>Yes</td>
<td>0</td>
<td>78.18</td>
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<tr>
<td>18</td>
<td>MW 10-P</td>
<td>31</td>
<td>70</td>
<td>6 bulbs</td>
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<td>68.5</td>
<td>107.51</td>
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<td>41</td>
<td>97.86</td>
<td>1.37</td>
</tr>
<tr>
<td>19</td>
<td>MW 10-P*</td>
<td>31</td>
<td>70</td>
<td>6 bulbs</td>
<td>15</td>
<td>63.4</td>
<td>119.9</td>
<td>Yes</td>
<td>55</td>
<td>90.57</td>
<td>1.56</td>
</tr>
</tbody>
</table>

P: Plain, S: Spiral, Comb: Combination of plain and spiral, ID: Indented. *Cable bolt did not fail.
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Shear behaviour of cable bolts subjected to single shearing

Figure 5-17 Plain MW10 cable bolt test results

Figure 5-18 Spiral MW9 cable bolt test results

Figure 5-19 Secura HGC cable bolt test results
Chapter IV  
Shear behaviour of cable bolts subjected to single shearing

Figure 5-20 Plain SUMO cable bolt test results

Figure 5-21 Indented SUMO cable bolt test results

Figure 5-22 Indented TG cable bolt test results
Figure 5- 23 Un-bulbed plain Superstrand cable bolt with 15 t pre-tension load

Figure 5- 24 Plain Garford cable bolt with bulb spacing and 0 t pre-tension load

Figure 5- 25 Plain MW10 cable bolt test results (repeated tests)
5.3.1 Effects of pretension load

The application of pre-tension load had considerable influence on the performance of the cable bolt in shear, which was demonstrated for all samples tested. Figures 5-26 and 5-27 represent the effect of pretension load on the peak shear load and shear displacement at peak shear load of 12 cables. The load-displacement data shows that the axial load applied to the pre-grouted cables of various types provides greater stiffness to the system. Therefore, the elastic region of deformation reduces and a steeper slope is characterised as the displacement influenced by the cable stiffness. The hypothesis is that the peak shear load decreases by pre-tension due to increasing the stiffness and reducing the flexibility of cable bolt. Data showed that displacement at peak shear load occurred at a lesser value when comparing the pre-tensioned cable with 0 t pre-tension strand counterpart.

Also, the presence of the indentation in the cable bolt surface profile decreased the shear displacement of the cable bolt during shearing. The surface roughness of the wires increased due to indentation, which has created higher interlocking bonding at the cable-grout interface. Thus, the indented cable bolts returned the lowermost displacements at peak shear load of all the tests; however this was not the case with plain cables because when the cables were found to debond, it increased in displacement. Obviously, the increased shear displacement in plain cables was attributed to cable debonding as shown by LVDT monitoring results in Appendices A.
Chapter IV  

Shear behaviour of cable bolts subjected to single shearing

Figure 5-26 Effects of pretension loads on peak shear load of cable bolts. Note: Plain SUMO cable was debonded

Figure 5-27 Shear displacement at peak shear loads for all tests
5.3.2 Effect of cable profile geometry

Figures 5-28 and 5-29 show the effect of cable bolts profile geometry on the shear strength of cable bolts. Figure 5-28 shows the comparison between shear performances of various cable bolts at 0 t pre-tension. The Secura HGC and plain MW10 achieved the highest shear load strength of all cable types. Plain SUMO cable achieved the third highest in shear strength. The comparison between the performance of plain MW10 and spiral MW9 and also plain and indented SUMO indicates that the plain cable bolt had higher shear strength compared with spiral irrespective of the cable type. This is because of mass reduction due to indentation. One important factor in higher result of plain MW10 and Secura HGC was having 600 mm and 500 mm lay length and greater wire diameters compared with other types of cable bolt. Also, these two cable types have higher UTS. On the other hand, the comparison between the performances of cable bolts in 15 t pre-tension appeared to suggest that the plain MW10 cable without bulb structure and the plain SUMO cable had greater shear strength (Figure 5-29). Also, no consideration is given to the effect of debonding on the peak shear load, which in general leads to higher values. This issue will be discussed later in this chapter.

Figure 5-28 Effects of cable bolt profile geometry at 0 t pretension
5.3.3 Effect of birdcage on the peak shear load of cable bolts

The comparison between tests 1 and 3 explores the effect of bulb shaped structures on plain MW10 cable bolt with 15 t pre-tension loads. The plain MW10 with no bulb shaped structures reached 68.34 t peak shear load, however with six bulb shaped structures it reached 60.39 t. The plain cable bolts with bulb structure showed lower shear strength (about 11.6% less) rather than those of without bulbs (Figure 5-30).

Results of tests 5 and 6 compares the effect of bulb structure on the spiral MW9 cable bolt. The peak shear load achieved by spiral MW9 with and without bulb structures were 43.93 t and 49.7 t. This means that the effect of bulb structure on the spiral MW9 was decreasing the peak shear load by 11.6% (completely similar to plain MW10).

Therefore, it was observed that bulb structure had a negative impact on the peak shear strength of cable bolt in single shear test due to a rigid structure provides by bulb (Figure 5-30). However the role of bulb structure in this programme of testing was
limited as the number of tests undertaken was not sufficient to draw any meaningful conclusion. Further studies on the plain MW10 cables were also inconclusive as the cable did not snap.

![Image of bar chart](image)

**Figure 5-30** Effects of birdcage on peak shear load of cable bolts

### 5.3.4 Cable bonding and debonding

As previously mentioned, the dial gauges at each extremity of the cable, strain gauges (for plain MW10 and spiral MW9) and centrally located LVDTs were installed to study bonding and debonding at the cable-grout interface.

The results of the study demonstrated that all plain wire strands were debonded irrespective of the pretension loads and bulbing. The debonded cables were plain MW10, plain SUMO and plain Superstrand. From the debonded cables, only plain MW10 cables were fitted with strain gauges and hence concern was raised as a possible reason for the cable debonding. Figures 5-31 represents the results obtained from strain gauges (First three graphs) and LVDT (last graph) for un-bulbed plain MW10 with 15 t of pretension subjected to shearing. It is noted that strain gauges of 1, 2 and 3 recorded...
debonding at 27, 22 and 21 mm shear displacements respectively. Debonding in the plain cable bolts occurred due to insufficient embedment length of encapsulation in each of 1.8 m long per side. This means that more than 1.8 m of encapsulation on each side is required to prevent the plain cables from debonding. These values are close to the displacement (20 mm) corresponding to debonding measured by LVDT (i.e. end movement).

The results also showed that the presence of strain gauges on the spiral MW9 cable bolts did not cause the cable to debonding as shown in Figure 5-32 for bulbed spiral MW9 with 15 t of pretension. This can be justified by the interlocking provided by the surface appearance of the spiral cable bolts during shearing. The result of bonding and debonding of all tested cable bolts are shown in Appendices A.

Figure 5-31 Strain gauges and LVDT measurement for the plain MW10 cable bolt with 15 t of pretension load and without bulbs
Figure 5-32 Strain gauges measurement for spiral MW9 cable bolts with 15 t of pretension load and without bulbs

It is also worth noting that as the cable debond, the excessive protruding end of the cable on the debonded side undergoes deformation as shown in Figure 5-33, which may offer the inaccuracy of the LVDTs reading.

It should be recognised that shear testing of cable does not constitute a realistic way of assessing cable deboning. Realistically pull testing is the method of assessing cable debonding.

Figure 5-33 Bending in plain MW10 cable bolt with 0t pretension load during the test

5.3.5 Cable bolt’s mode of failure

The sample was retrieved from the shearing instrument after shear testing and the mode of the failure was visually examined. As shown in Figure 5-34, the failure mode was
mostly a combination of shear/tensile and tensile failure. The characteristic of shearing rig was that the left side of the tested rig was fixed and did not move vertically, leaving the right side to be moved vertical down to shear. The presence of a ‘cup’ and ‘cone’ on some wires of the strand suggests that the wires failed in tension the other wire failures with smooth angle shows the combination of tensile and shear failure. Table 5-2 show the calculated result of the failure mode in cable bolts.

Figure 5-34 Cable bolts failure in the right side of shearing directions
Table 5-2 Tensile and shear failure area in wires of cable bolts

<table>
<thead>
<tr>
<th>Test No.</th>
<th>No. of wires failed in tension</th>
<th>No. of wires failed in shear</th>
<th>No. of wires failed in tension and shear</th>
<th>Failure area in tension (mm$^2$)</th>
<th>Failure area in shear (mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>2</td>
<td>6</td>
<td>192.4</td>
<td>192.4</td>
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<td>7</td>
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<td>3</td>
<td>7</td>
<td>0</td>
<td>115.5</td>
<td>269.4</td>
</tr>
<tr>
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<td>4</td>
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<td>76.93</td>
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<td>NA.</td>
<td>NA.</td>
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<td>NA.</td>
<td>NA.</td>
<td>NA.</td>
<td>NA.</td>
<td>NA.</td>
</tr>
</tbody>
</table>

5.3.6 Effect or rate of loading on the test results

The rate of loading is an important factor which may affect the validity of test results. The shear rate was subjected to cable bolts in double shearing was 1 mm/min however the shearing was applied to the samples manually and non-constant in single shear test which may affect the result. Table 5-1 showed the rate of loading in tests. It is evident that the rate of loading was varied between 1.37 and 38 which consists of a wide range
of loading. The excessive rate of loadings was applied to samples, particularly in tests 1, 2, 4 and 11. The high rate of loading from the extreme acceleration of the hydraulic pressure applies shock loading to the sample. Creep is the rheological aspect of the material due to the gradual rate of loading or mechanical stresses to deform the material permanently which decreases its strength. When the rate of loading is high, the sample fails with less creep and the final shear load might be higher compared with the sample with lower shear rate. Therefore, it is suggested to use a controlled system to fix the amount of shear rate as desirable value and generally less than 4 mm/min similar to the tests carried in double shear reported by Aziz et al. (2015a, 2015b).

5.4 Comparison of the double shear and single shear test results

This section draws together the data sets covered previously, which are a comparison between results of single shear and double shear tests. The main aim of this research was to study effects of various parameters including cable type, cable surface profile, birdcage and pre-tension load on the shear behaviour of cable bolts, which was achievable by comparing results of MKIII (no contacts between concrete joint shear faces) and single shear tests.

There are variables between these two sets of tests. Firstly, no debonding was observed in the double shear tests due to the use of barrel and wedge although the single shear apparatus was capable of determining bonding and debonding in various cable surface profiles. Secondly, the double shear assembly was unconfined which is in contrast with the confined single shear assembly. The cable bolts have confinement in in situ situation; thus, the single shear tests replicate the field condition better. On the other hand, the rate of shearing was inconstant in the single shear due to manual application
of vertical shear load although the rate of shear was fixed at 1 mm/min in double shear tests. High shear rate might affect the shear load results.

Although there were some differences in the test conditions between the single and double shear tests results, their comparison is worthy of consideration. Table 5-3 shows the result of similar cable types with similar pre-tension loads subjected to both double and single shear tests. The peak shear loads in double shear tests were divided by two to achieve the shear load on one face.

As Table 5-3 shows, there were some differences between the results obtained from the single shear and double shear tests for plain SUMO cable with 0 t pre-tension load. This was because the cable did not fail in 100 mm of displacement in double shear test and it did not reach its peak shear load. Therefore, the peak shear load in double shear test had to be greater than 443 kN and more likely to be closer to 558 kN as obtained from single shear test.

Also, there were significant differences between the result of single shear and double shear tests for plain SUMO with 15 t pretension load and un-bulbed plain MW10 cable with 15 t pre-tension load. The main reason was because of debonding in single shear tests for smooth surface profile cables. To remove debonding from the plain profile cable, the single shear apparatus embedment length should be increased to gain similar results to the double shear tests.

On the other hand, the result of rough surface profile cable bolts were similar with each other because no debonding was observed in indented and in spiral cable bolts. The result achieved for indented SUMO with 0 t pretension load showed less than 1% difference in the result. The comparison between results of indented SUMO in 15 t pretension load in double shear and single shear tests were 383 kN and 373 kN.
respectively, which means 2% differences in the results. Finally, the results of un-bulbed spiral MW9 cable bolts had 18.6% variance but may not be accepted as the limited number of tests was undertaken.

The rate of loading might be another reason of increasing peak shear loads in the single shear test due to the shock loading in the system. The plain SUMO, un-bulbed plain MW10 and un-bulbed spiral MW9 achieved higher results in single shear compared with double shear test. The rate of loading was not the main reason of this variance however it is countable.

Table 5-3 Comparison of the double and single shear tests results

<table>
<thead>
<tr>
<th>Cable type/ pretension load (kN)</th>
<th>Double shear test (MKIII)</th>
<th>Single shear test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak shear load (kN)</td>
<td>Displacement at peak shear load (mm)</td>
</tr>
<tr>
<td>Plain SUMO/ 0</td>
<td>443*</td>
<td>100</td>
</tr>
<tr>
<td>Plain SUMO/ 150</td>
<td>426</td>
<td>88.2</td>
</tr>
<tr>
<td>ID SUMO/ 0</td>
<td>407.5</td>
<td>93.4</td>
</tr>
<tr>
<td>ID SUMO/ 150</td>
<td>383</td>
<td>85.7</td>
</tr>
<tr>
<td>UB Plain MW10/ 150</td>
<td>462</td>
<td>88.5</td>
</tr>
<tr>
<td>UB Spiral MW9/ 150</td>
<td>419</td>
<td>88.47</td>
</tr>
</tbody>
</table>

* Sample did not fail
Figure 5-35 shows the comparison between the test results of the double shear (MKIII) and single shear tests which shows the good agreement in rough surface profile cable bolts however there is a big variance between results in smooth surface cable bolts.

![Figure 5-35](image.png)

**Figure 5-35** Comparison of single shear and double shear test without friction surfaces (MKIII)

### 5.5 Summary

19 single shear tests were conducted on the 3.6 m concrete blocks to study the effect of cable type, surface profile type, pre-tension load, birdcage structure, bonding and debonding, and the failure mode of cable bolts.

It should be recognised that shear testing of cable does not constitute a realistic way of assessing cable deboning. Realistically pull testing is the method of assessing cable debonding.
Thus, it can be inferred that:

- Plain cable bolts have higher peak shear load compared with indented cable bolts irrespective of the cable type. This is because of the relatively greater cross sectional area of the plain wires of the strand.
- Plain MW10 is superior solely in terms of shear load resisted which is due to greater UTS because of increased numbers of strand wires to ten, plain surface profile type and lay length.
- Plain wired strand cables have debonded for the given cable encapsulation length of 1.8 m. All debonded cables did not fail or snap. Thus, it is recommended to increase the embedment length or increase encapsulation length to avoid debonding.
- Debonded cable leads to increase displacement at peak shear load.
- No debonding was observed in spiral and indented cable bolts because of having higher interlocking in the cable/grout interface.
- The failure modes in the cable bolt are the combination of pure tensile and tensile/shear failures. No strand wire failed in pure shear.
- Increased displacement influenced the nature of sheared wires of the strand. Generally, wires fail closer to shear/tensile state in comparison to failure in tension. Increased cable displacement causes cable wires fail mostly in tension.
- In general, the failed strand peak shear load was lower with increased pretension load in un-debonded condition in shear. Higher pretension load causes the cable to stiffen and fail with lower vertical shear displacement.
The role and influence of using bulbs was inconclusive because of the limited number of tests undertaken in this study. This study should be further evaluated with more tests.

No realistic conclusion can be drawn about the effect of a single parameter on the shearing behaviour. The variable parameters considered are cable lay length, bulbing, rate of loading, wire smoothness (plain) or roughness (indented or spiral).

It appears that reducing the rate of loading provided consistent results. It is accepted that the rate of loading of less than 4 mm/min is a reasonable rate of testing rate.

The result of double shear tests with no friction is compatible with single shear tests for rough surface cable bolts. There are some variances in the test results due to different test methodologies and rate of shearing.

The result of double shear tests with no friction is not compatible with the single shear tests for smooth surface cable bolts due to debonding and high rate of shear in single shear tests.
6.1 Introduction

The performance of cable bolts in shear is a crucial and challenging factor in civil engineering tunnelling design and underground mine excavations. Failure occurs when the strength of cable bolts is less than the load applied to the system. Thus, understanding the performance and strength of cable bolts in shear is an important element for safe design. There are two main approaches to quantify the mechanical properties of cable bolts, namely, theoretical and empirical. The theoretical approach incorporates the Fourier series, energy balance theory and tri-linear methodology while the empirical approach relies on the analysis of experimental data. This study follows up the numerical tools to perform studies based on laboratory tests and theoretical assumptions. The Fast Lagrangian Analysis of Continua (FLAC 2D) was able to simulate the shear performance of fully grouted cable bolts subjected to double and single shear testing and British Standards method. An embedded program (FISH) was equipped in the FLAC 2D to offer a wide range of applications to users.

6.2 Analytical modelling of performance of pre-tensioned fully grouted cable bolts subjected to double shearing

This part of the study contains the mathematical simulation of the shear behaviour of cable bolts subjected to double shear tests.
6.2.1 Requirements of new mathematical models

Various models for predicting the shear strength of cable bolts and rock bolts under tensile and shear loadings were reviewed in Chapter III however there was a lack of information on the analytical simulation of the shear performance of cable bolts.

As previously mentioned in Chapter III, Aziz et al. (2015b) proposed a mathematical model to determine the shear behaviour of cable bolts using a combination of Mohr Coulomb criterion and Fourier series scheme.

The proposed model by Aziz et al. (2015b) is:

\[
\tau_p = \left( \frac{a_0}{2} + \sum_{n=1}^{A} a_n \cos\left( \frac{2n\pi}{T} \cos^{-1}\left( \frac{-4a_2^2 + \sqrt{16a_2^4 - 48a_2a_4 + 144a_4^2}}{24a_4} \right) \right) \right) \tan(\phi) + c \quad (6.1)
\]

where, \( \tau_p \) is the shear stress and \( S \) is the shear load, \( c \) is cohesion, \( a_n \) is Fourier coefficient, \( n \) is the number of Fourier coefficient, which is considered between 0 and 3, \( u \) is the shear displacement and \( T \) is the shearing length.

This model is only capable of determining the peak shear strength of the cable bolt in double shearing. The limitation of this model is on simulating the shear performance of cable bolt at different stages of behaviour. Thus, it is important to propose a model to simulate the shear performance of cable bolts in different stages.

6.2.2 Different stages of shear performance of cable bolts

The shear behaviour of a fully grouted cable bolt has different stages. The first stage is the elastic stage where deformation is temporary and the system deforms because of the load applied to the system and recovers and returns to its original shape once the external load is removed. After the elastic stage, the system enters into the non-linear
stage, and plasticity occurs because the deformation in the system is non-reversible and permanent. Subsequently, the softening which is part of the plastic stage occurs until the cable reaches the peak shear load. After this stage, the strands of the cable bolt break and the shear strength decreases until the residual point where the system fails.

Figure 6-1 shows applied loads to cable bolts during shearing tests. S, N and i in this figure represent the shear load, axial load and the rotation angle of cable respectively. The shear load using the 500 t compression machine is applied vertically to the sample. The axial load is gradually generated along the cable increasingly as it is being subjected to shearing. During the test, the cable bolt bends with the applied shear load and the formation of i to the horizontal axis.

![Diagram of loads applied to the cable bolt](Image)

**Figure 6-1** Loads applied to the cable bolt

### 6.2.2.1 Elastic stage

The relationship between the shear load and shear displacement in the elastic stage is linear and derived as:

\[ S = K \nu \]  

(6.2)
where, $S$ is the shear load, $K$ is the shear stiffness depending on the cable type and $\nu$ is shear displacement.

**6.2.2.2 Strain-softening stage**

After the elastic stage, the strain-softening (elasto-plastic or non-linear) stage begins, which can be described as the transitional stage between elastic and failure zones where the shear behaviour of the fully grouted cable bolt is still linear as shown in Figure 6-2. The shear stiffness here is significantly smaller than the elastic stage and therefore the slope of the strain softening stage is smaller than the elastic stage. The strain softening stage happens because the cable strands start to yield. The cable deformation will be permanent in this stage. Figure 6-2 shows how the slope of shear load versus shear displacement decreases from the elastic stage to strain softening stage.

$$S = \beta K \nu$$  \hspace{1cm} (6.3)

$\beta$ is the reduction factor due to strain softening stage and can be derived as:

$$\beta = \frac{\tan \nu}{\tan \theta}$$  \hspace{1cm} (6.4)
Table 6-1 shows the reduction factor ($\beta$) for the strain softening stage compared with the elastic stage. It shows the slope of the softening stage is averaging 42% of the elastic stage and changes between 30 and 60%. This occurs because the cable strands start to yield after the elastic stage and the shear load decreases.

Table 6-1 Reduction factor in hardening stage to elastic stage

<table>
<thead>
<tr>
<th>Cable type/ Pretension</th>
<th>Line slope (Elastic)</th>
<th>Line slope (strain hardening)</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spiral Superstrand/ 25 t</td>
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<td>8.19</td>
<td>0.29</td>
</tr>
<tr>
<td>Plain Superstrand/ 25 t</td>
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<td>9.83</td>
<td>0.36</td>
</tr>
<tr>
<td>ID TG/ 25 t</td>
<td>18.58</td>
<td>8.09</td>
<td>0.44</td>
</tr>
<tr>
<td>ID SUMO/ 25 t</td>
<td>24.48</td>
<td>8.61</td>
<td>0.35</td>
</tr>
<tr>
<td>ID SUMO/ 10 t</td>
<td>16.75</td>
<td>9.07</td>
<td>0.54</td>
</tr>
<tr>
<td>Plain SUMO/ 25 t</td>
<td>26.95</td>
<td>9.78</td>
<td>0.36</td>
</tr>
<tr>
<td>Plain SUMO/ 10 t</td>
<td>14.23</td>
<td>8.65</td>
<td>0.61</td>
</tr>
<tr>
<td>Garford twin-strand/ 0 t</td>
<td>19.35</td>
<td>5.57</td>
<td>0.29</td>
</tr>
<tr>
<td>Secura HGC/ 25 t</td>
<td>21.78</td>
<td>7.66</td>
<td>0.35</td>
</tr>
<tr>
<td>Secura HGC/ 10 t</td>
<td>12.73</td>
<td>7.58</td>
<td>0.60</td>
</tr>
</tbody>
</table>

6.2.2.3 Peak shear load to residual

The shear behaviour of a cable bolt at the peak load and after is not linear and the proposed model in this stage is developed using the energy balance theory. The axial load is not constant because of the characteristics of the test.; therefore, the axial load is modelled incorporating the Fourier series. Fourier series is a mathematical function applied in a large variety of engineering problems mainly adopting the principle of superposition for the rigid body deformation.
6.2.2.3.1 Fourier series

The Fourier series is used to replicate the wave as the sum of simple sine waves. The graph of the axial load versus shear displacement is similar to the sine wave from 0 to 90 degrees. The Fourier series concept, as described below, is incorporated to simulate the variation of axial load against shear displacement.

\[ N = \frac{a_0}{2} + \sum_{n=1}^{\infty} \left[ a_n \cos\left(\frac{2n\pi u}{T}\right) + b_n \sin\left(\frac{2n\pi u}{T}\right)\right] \]  \hspace{1cm} (6.5)

\[ a_n = \frac{2}{T} \int_0^T \sigma_n \cos\left(\frac{2n\pi u}{T}\right) du \]  \hspace{1cm} (6.6)

\[ b_n = \frac{2}{T} \int_0^T \sigma_n \sin\left(\frac{2n\pi u}{T}\right) du \]  \hspace{1cm} (6.7)

where, \(a_n\) and \(b_n\) are Fourier coefficients, \(n\) is the number of Fourier coefficient, \(u\) is the shear displacement and \(T\) is the shearing length. For simplification, \(b_n\) is considered as zero.

The axial load (N) is calculated by the following equation and using \(a_0\) to \(a_3\). Thus:

\[ N = \frac{a_0}{2} + a_1 \cos\left(\frac{2\pi u}{T}\right) + a_2 \cos\left(\frac{4\pi u}{T}\right) + a_3 \cos\left(\frac{6\pi u}{T}\right) \]  \hspace{1cm} (6.8)

6.2.2.3.2 Energy Balance theory

The energy balance theory incorporates the first law of thermodynamics. This theory states that energy in a system is always constant. The energy crosses the boundary of the system and makes changes in the internal energy of the system in the exact similar amount. Based on this theory:

\[ \Delta U = Q - W \]  \hspace{1cm} (6.9)
where, $\Delta U$ is the change in the internal energy of system, $Q$ is the net energy crossing boundary and $W$ is the net work done by or on the system.

In double shearing, the energy applied to the system is the consequence of both shear and axial loads. When the shear load is applied to the system and the cable bends, the axial load has two different components of $N \sin i \, dv$ and $N \cos i \, du$ in vertical and horizontal directions respectively (Figure 6-1). $N \sin i \, dv$ is in the same direction as the shear load and it is a part of the energy added to the system, while $N \cos i \, du$ is against shear energy and is an internal energy. The external energy applied to the system changes the internal energy in the system. The energy is used for breaking the strands of the cable bolt. The breakage of the cable is mostly the combination of failure in tension and shear (Figure 6-3). Therefore, $A_e$ shows the area of cable wires breaking in shear, while $A_t$ is the area due to tensile failure. The energy used as a friction between concrete blocks joints contact surfaces is less compared with the high amount of energy consumed in cable breakage. This is the reason that the energy dissipated for friction between the concrete blocks in the joints did not considered in the model directly however this friction affects the axial load generated in cable bolt as well. Thus, the Fourier series coefficients determined includes the effect of friction. It is worth noting that if the concrete blocks are not in contact, the Fourier coefficients will be different and results of axial and shear loads simulations will be different and smaller compared with current results. Also, no debonding was observed in the contact surface of cable bolts and the grout. Equation 6.10 show the use of energy balance theory in double shearing system.

\[
S \, dv - N \cos i \, du + N \sin i \, dv - \sigma_y \, du \, A_t - \sigma_s \, dv \, A_s = 0
\]  \hspace{1cm} (6.10a)
where, $S$ is shear load, $N$ is axial load, $dv$ is shear displacement, $du$ is axial displacement, $\sigma_y$ is Yield strength of cable bolt, $\sigma_s$ is shear strength of the cable bolt, $A_s$ is the broken area of cable in shear, and $A_t$ is the broken area of cable in tension.

By dividing the equation to $dv$ and considering $\cot i = du/dv$, equation 6.11 is derived as:

$$S - N \cos i \cot i + N \sin i - \sigma_y A_t \cot i - \sigma_s A_s = 0$$

(6.10b)

$$S = N(\cos i \cot i - \sin i) + \sigma_y A_t \cot i + \sigma_s A_s$$

(6.10c)

Therefore, the shear load in the plastic stage is derived from equations 6.8 and 6.10.

$$S = \left[ \frac{a_0}{2} + a_1 \cos \left( \frac{2\pi u}{T} \right) + a_2 \cos \left( \frac{4\pi u}{T} \right) + a_3 \cos \left( \frac{6\pi u}{T} \right) \right] (\cos i \cot i - \sin i) + \sigma_y A_t \cot i + \sigma_s A_s$$

(6.11)

As discussed previously, the shear strength of the cable, in the elastic and plastic stages, can be determined using Equations 6.2, 6.3 and 6.11; therefore:

1. $S_{\text{elastic}} = K v$

(6.12a)

2. $S_{\text{strain softening}} = \beta K v$

(6.12b)

3. $S_{\text{failure}} = \left[ \frac{a_0}{2} + a_1 \cos \left( \frac{2\pi u}{T} \right) + a_2 \cos \left( \frac{4\pi u}{T} \right) + a_3 \cos \left( \frac{6\pi u}{T} \right) \right] (\cos i \cot i - \sin i) + \sigma_y A_t \cot i + \sigma_s A_s$

(6.12c)
Figure 6-3 Breakages of cable wires in shear and tension

The values of $a_0$, $a_1$, $a_2$ and $a_3$ were calibrated experimentally on tests conducted on various cables under different pretension loads as shown in Table 6.2.

Table 6-2 Fourier coefficients of the tests

<table>
<thead>
<tr>
<th>Cable type/ Pretension</th>
<th>$a_0$</th>
<th>$a_1$</th>
<th>$a_2$</th>
<th>$a_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spiral Superstrand/ 25 t</td>
<td>624.59</td>
<td>-56.76</td>
<td>-31.5</td>
<td>22.96</td>
</tr>
<tr>
<td>Plain Superstrand/ 25 t</td>
<td>616.62</td>
<td>-10.95</td>
<td>-80.13</td>
<td>62.67</td>
</tr>
<tr>
<td>Indented TG/ 25 t</td>
<td>636.82</td>
<td>-67.87</td>
<td>-14.88</td>
<td>18.57</td>
</tr>
<tr>
<td>ID SUMO/ 25 t</td>
<td>387.59</td>
<td>83.53</td>
<td>-36.68</td>
<td>-5.05</td>
</tr>
<tr>
<td>ID SUMO/ 10 t</td>
<td>335.31</td>
<td>-27.32</td>
<td>-62.84</td>
<td>40.02</td>
</tr>
<tr>
<td>Plain SUMO/ 25 t</td>
<td>534.76</td>
<td>3.47</td>
<td>-75.64</td>
<td>55.49</td>
</tr>
<tr>
<td>Plain SUMO/ 10 t</td>
<td>449.78</td>
<td>-136.34</td>
<td>16.39</td>
<td>-3.91</td>
</tr>
<tr>
<td>Garford twin-strand/ 0 t</td>
<td>191.25</td>
<td>-146.16</td>
<td>70.55</td>
<td>-26.46</td>
</tr>
<tr>
<td>Secura HGC/ 25 t</td>
<td>731.58</td>
<td>-114.83</td>
<td>57.25</td>
<td>-25.95</td>
</tr>
<tr>
<td>Secura HGC/ 10 t</td>
<td>646.722</td>
<td>-176.63</td>
<td>30.09</td>
<td>-6.84</td>
</tr>
</tbody>
</table>
Figure 6-4 shows the breakage and deformation angle of cable bolts. The deformation angle for all cables is between 26 and 29 degrees. The values of the rotation angle show that the shear displacement and normal displacement are mostly constant for various cables. As previously mentioned, most of the energy dissipated in the system occurs because of the cable wires breaking. This breakage can be in shear or tension. The yield strength of the strand in shear and tension is different. Therefore, it is obvious that considering the area of the cable which is subjected to shear and tensile breakage is important. Table 6-3 shows the amount of cable breakage in shear and tension. Another important factor shown in this table is the deformation angle in the cable.

Figure 6-4 Deformation angles of the shearing test conducted on the fully grouted cable bolts
### Table 6-1 Amount of cable wire breakage in shear and tension

<table>
<thead>
<tr>
<th>Test</th>
<th>As (mm$^2$)</th>
<th>A (mm$^2$)</th>
<th>$i$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indented Superstrand/ 25 t pretension load</td>
<td>212.58</td>
<td>81.83</td>
<td>28.12</td>
</tr>
<tr>
<td>Plain Superstrand/ 25 t pretension load</td>
<td>35.33</td>
<td>270.24</td>
<td>27.47</td>
</tr>
<tr>
<td>Indented TG/ 25 t pretension load</td>
<td>346.18</td>
<td>0</td>
<td>28.62</td>
</tr>
<tr>
<td>Indented SUMO/ 25 t pretension load</td>
<td>0</td>
<td>346.19</td>
<td>26.58</td>
</tr>
<tr>
<td>Indented SUMO/ 10 t pretension load</td>
<td>153.86</td>
<td>192.33</td>
<td>26.26</td>
</tr>
<tr>
<td>Plain SUMO/ 25 t pretension load</td>
<td>38.46</td>
<td>307.72</td>
<td>26.86</td>
</tr>
<tr>
<td>Plain SUMO/ 10 t pretension load</td>
<td>0</td>
<td>346.18</td>
<td>27.31</td>
</tr>
<tr>
<td>Garford twin-strand/ 0 t pretension load</td>
<td>102.14</td>
<td>183.86</td>
<td>26.74</td>
</tr>
<tr>
<td>Secura HGC/ 25 t pretension load</td>
<td>76.93</td>
<td>269.26</td>
<td>27.01</td>
</tr>
<tr>
<td>Secura HGC/ 10 t pretension load</td>
<td>0</td>
<td>346.19</td>
<td>28.95</td>
</tr>
</tbody>
</table>

Figures 6-5 to 6-14 show the comparison between measured test data and model predicted results for the axial load versus shear displacement by using Fourier series for modelling. This comparison shows how perfectly the Fourier series can model the axial load versus shear displacement for different cable bolts subjected to different pretensioning.

The results from this research indicate that the plain cable has higher peak shear strength when compared with the indented cable with the same type of cable bolt, same grout type and pre-tensioning. The reason of this difference is that because of the indentation, the indented strands have a smaller cross sectional area. The result of the tensile tests for a wire of plain and indented cables shows the similarity to the shear test. Also, no rotation of cable bolt was observed in the shear test. Figures 6-5 to 6-14 represent the model proposed for the shear performance of cable bolts in comparison to the values of test data. The shear test graph of the test data consists of the elastic stage, strain-softening and peak shear load up to the residual point. After the peak shear load,
the drops in the shear load are observed when the cable strands break. The drop in the shear load is higher when the diameter of the strand is larger.

**Figure 6-5** Shear load modelling of spiral Superstrand with 25 t pretension load

**Figure 6-6** Shear load modelling of plain Superstrand with 25 t pretension load
Figure 6-7 Shear load modelling of indented TG with 25 t pretension load

Figure 6-8 Shear load modelling of indented SUMO with 25 t pretension load
Figure 6-9 Shear load modelling of indented SUMO with 10 t pretension load

Figure 6-10 Shear load modelling of plain SUMO with 25 t pretension load
Figure 6-11 Shear load modelling of plain SUMO with 10 t pretension load

Figure 6-12 Shear load modelling of Garford twin-strand with 0 t pretension load
Figure 6-13 Shear load modelling of Secura HGC with 25 t pretension load

Figure 6-14 Shear load modelling of Secura HGC with 10 t pretension load
An important factor in the shear behaviour of cable bolts is pre-tensioning. From Equation 6.12, it is inferred that pre-tensioning affects the shear strength of cable bolts; however, it is obvious that there are different related factors such as cable type, the deformation angle of cable and pre-tensioning that could affect the shear load.

Results indicate a good agreement between test results and modelling simulations.

### 6.3 Analytical modelling of performance of pre-tensioned fully grouted cable bolts subjected to single shearing

In this section, a mathematical constitutive model for the shear behaviour of cable bolts is proposed by modifying the tri-linear model developed by Wu et al. (2009). The original tri-linear model was proposed to measure the relationship between the shear stress ($\tau$) and slip ($\delta$) at the rod-grout interface in pull-out tests which is shown in Figure 6-15 and Equation 6.13.

![Figure 6-15 Relationship between interfacial shear stress and slip (Wu et al., 2009)](image)

$$\tau = k\delta \quad (0 \leq \delta \leq \delta_1) \quad (6.13a)$$
\[ \tau = \frac{\tau_p \delta_2 - \tau_f \delta_1}{\delta_2 - \delta_1} - k \frac{\tau_p - \tau_f}{\delta_2 - \delta_1} \left( \delta_1 \leq \delta \leq \delta_2 \right) \]

\[ \tau = \tau_f \quad \delta > \delta_2 \]  

(6.13b)  

(6.13c)

where, \( \tau_u \) is the shear strength at the rod-grout interface, \( \tau_s \) is the residual frictional stress at the rod-grout interface, \( \tau \) is shear stress, \( \delta \) is slip at the rod-grout interface, \( \delta_1 \) is slip at peak bond strength, \( \delta_2 \) is slip at bond failure and \( k \) is coefficient factor.

The single shear test consists of three stages of elastic, strain softening and failure, which can be simulated using the modified tri-linear concept as plotted in Figure 6-16. Stage (I) shows the elastic stage where the encapsulated cables bolt will return to its original position upon unloading. After yielding, the shear behaviour will be in the strain softening in which the shear strength increases monotonically based on a stiffness value lower than that of elastic stiffness. The shear strength then diminishes to the residual value once peak shear strength is exceeded. Thus, the shear strength of the encapsulated cable bolts can be summarised in three stages as proposed in Equation 6.14.

**Figure 6-16** Relationship between shear stress and shear displacement in single shear test
\[ \tau = k \delta \quad (0 \leq \delta \leq \delta_1) \]  

(6.14a)

\[ \tau = \beta \delta + c \quad (\delta_1 \leq \delta \leq \delta_2) \]  

(6.14b)

\[ \tau = \text{peak shear load to } 0 \quad (\delta \geq \delta_2) \]  

(6.14c)

where, \( \tau \) is the shear strength of encapsulated cable bolt, \( \tau_y \) is shear load at yield point, \( \tau_p \) is peak shear load, \( \tau_r \) is residual shear load, \( \delta_1 \) is shear displacement at yield point, and \( \delta_2 \) is shear displacement at peak shear load.

The model coefficients were calibrated according to the collected experimental data for 10 tests with various cable types and pretension values as listed in Table 6-4. Figures 6-17 and 6-18 show the experimental and model simulated results for different tests. Figure 6-19 shows the comparison between the peak shear load of experimental results and model simulation which shows a good agreement.

**Table 6-2** Modified tri-linear model coefficients

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Cable</th>
<th>Pretension load (t)</th>
<th>Bulb spacing (mm)</th>
<th>( kx+m )</th>
<th>( \beta \delta + c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Plain MW10</td>
<td>15</td>
<td>-</td>
<td>4.612x</td>
<td>0.6803\delta+6.8149</td>
</tr>
<tr>
<td>2</td>
<td>Plain MW10</td>
<td>0</td>
<td>500</td>
<td>5.094x</td>
<td>0.9376\delta+3.2584</td>
</tr>
<tr>
<td>3</td>
<td>Plain MW10</td>
<td>15</td>
<td>500</td>
<td>3.2209x</td>
<td>1.032\delta+2.4597</td>
</tr>
<tr>
<td>4</td>
<td>Spiral MW9</td>
<td>0</td>
<td>500</td>
<td>6.2385x</td>
<td>0.9521\delta+5.8935</td>
</tr>
<tr>
<td>5</td>
<td>Spiral MW9</td>
<td>15</td>
<td>500</td>
<td>3.127x</td>
<td>1.2054\delta+3.8939</td>
</tr>
<tr>
<td>6</td>
<td>Spiral MW9</td>
<td>15</td>
<td>-</td>
<td>4.6871x</td>
<td>1.0709\delta+5.488</td>
</tr>
<tr>
<td>7</td>
<td>Secura HGC</td>
<td>0</td>
<td>-</td>
<td>2.9341x-5.867</td>
<td>1.1989\delta+1.8909</td>
</tr>
<tr>
<td>8</td>
<td>Secura HGC</td>
<td>15</td>
<td>-</td>
<td>5.937x-10.923</td>
<td>1.1418\delta+4.8261</td>
</tr>
<tr>
<td>9</td>
<td>ID SUMO</td>
<td>0</td>
<td>500</td>
<td>2.033x-2.6676</td>
<td>0.9258 \delta+1.5104</td>
</tr>
<tr>
<td>10</td>
<td>ID SUMO</td>
<td>15</td>
<td>500</td>
<td>3.043x-5.6936</td>
<td>0.9279 \delta+6.9323</td>
</tr>
</tbody>
</table>
Figure 6- 17 Results of Tests 1 to 10

Figure 6- 18 Modelling simulation for Tests 1 to 10
6.4 NUMERICAL ANALYSIS

Fast Lagrangian Analysis of Continua (FLAC) is a two dimensional numerical modelling software used by geotechnical, civil, and mining engineers for advanced geotechnical analysis of soil, rock, groundwater, and ground support. FLAC 2D was used in this study to simulate:

1. The shear performance of fully grouted cable bolts subjected to double shear tests without contact (MKIII),
2. The shear performance of fully grouted cable bolts subjected to the British Standard test, and
3. The shear performance of fully grouted cable bolts subjected to single shear test.

FISH (a programming language embedded within FLAC) was used to determine the shear strength of the cable bolt as a function of relative shear displacement.

6.4.1 Plain Superstrand cable bolts subjected to double shear test

A plain Superstrand cable bolt subjected to double shear force when the concrete blocks are not in contact with each other was simulated by FLAC 2D. Figure 6-20
shows the schematic design of the fully grouted cable bolt by considering the boundary conditions, fixed points due to barrel and wedge and the load applied to the middle concrete blocks. The gap between concrete block surfaces in the experiment was created by using the bracing which is not shown in Figure 6-20.

![Figure 6-20 Schematic fully grouted cable bolt subjected to double shear test](image)

The grid was prepared with 105 grids in x direction and 30 grids in y direction. The block with dimensions of 1.05 x 0.3 m were created and then it was split into three blocks by defining two joints to represent the three concrete blocks of the double shear system. Then the cable bolt was introduced to the model with constant y value at 0.15 m and length of 1.05 m. The material properties of concrete blocks and the cable bolt were defined in FLAC. 105 segments with 106 nodes created for the cable bolt. Due to the usage of barrel and wedge in the experiment, nodes 1 and 106 were fixed. The FISH function was used in the next stage to measure the shear force created in the sample.

The UCS test had been conducted for the concrete block samples after 28 days and this value was measured as 40 MPa. Thus, \( f'_c \) is 40 MPa for concrete blocks. The elastic modulus of concrete block was calculated by:

\[
E = 4700 \sqrt{f'_c} = 29.72 \text{ GPa} \quad \text{(Wilde et al., 2012)}
\]
\[ \vartheta = 0.2 \]

The bulk modulus (k) and shear modulus (G) were determined by the following equations:

\[
k = \frac{E}{3(1-2\vartheta)} \quad (6.16)
\]

\[
G = \frac{E}{2(1+\vartheta)} \quad (6.17)
\]

The joints were only created by the null function in this simulation because there was no contact between concrete blocks and it was unnecessary to assign a model for the joints.

The following material properties were used in the model:

**Table 6-3** Model properties of the plain Superstrand cable bolt subjected to double shear test without contact

<table>
<thead>
<tr>
<th>Concrete block materials</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bulk modulus (k)</strong></td>
</tr>
<tr>
<td><strong>Shear modulus (G)</strong></td>
</tr>
<tr>
<td><strong>Density</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cable bolt properties</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Area</strong></td>
</tr>
<tr>
<td><strong>Second moment of Inertia</strong></td>
</tr>
<tr>
<td><strong>Yield load</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Grout properties</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Shear cohesion</strong></td>
</tr>
<tr>
<td><strong>Shear stiffness</strong></td>
</tr>
<tr>
<td><strong>Normal cohesion</strong></td>
</tr>
<tr>
<td><strong>Normal stiffness</strong></td>
</tr>
</tbody>
</table>
The material properties of concrete blocks, joints and cable bolt were accurately prescribed. Figure 6-21 shows the output from FLAC 2D software for simulation of fully grouted cable bolt before double shearing. Figure 6-22 shows how the FLAC 2D can simulate the shear load versus shear displacement for a plain Superstrand cable bolt subjected to double shearing without any contact between concrete blocks. This graph shows a good agreement between the model simulation and experimental data. It was observed that the peak shear loads were similar for the laboratory tests and the modelling simulation.

**Figure 6-21** Simulation of fully grouted cable bolt subjected to double shear test without contact between concrete block surfaces
Figure 6-22 Comparison between the shear load versus shear displacement for the experimental data and modelling simulation for double shear test without contact

Figure 6-23 shows that the peak shear loads that occurred in the joints for 20, 40 and 60 mm displacements were 198.1, 238 and 270.9 kN. The shear load was critical in the joint due to the bending and failure of cable bolts. For 80 mm displacement, the cable bolt failed but the highest amount of shear load was 210.9 kN, which was still in the joint. This graph also showed how the amount of shear load increased due to increasing shear displacement while shear load between joints decreased. The loading was symmetric and this was in agreement with the symmetric experimental situation and loading condition.

Figure 6-24 shows the vertical stress distributed in the concrete blocks after 100 mm displacement. The maximum stress was generated in the middle block particularly in the vicinity of joints. The extremity of the middle block concrete was subjected to excessive shear load due to vertical sheared cable displacement. It is evidently clear when the cable was sheared, the middle concrete block underwent compression in the top half while the lower part was in tension. This was reverse for corner blocks.
Figure 6-23 Shear load in the cable bolt for displacements of 20, 40, 60 and 80 mm for double shear test without contact. The x axis shows the shear displacement in mm, the y axis shows the shear load (The shear load unit is MN).

Figure 6-24 Shear stress contributions in the concrete blocks subjected to double shear tests without contact between concrete blocks in 100 mm displacement.
In order to prove the ability of FLAC 2D in simulation of cable bolts in double shear tests, more tests were simulated. Figures 6-25 and 6-26 include the comparison between the shear load versus shear displacement for the experimental data and FLAC modelling simulation for double shear tests without contact for indented and plain SUMO with 0 kN pretension load. Results from Figures 6-22, 6-25 and 6-26 show a good agreement between experimental and numerical simulation results.

**Figure 6-25** Comparison between the shear load versus shear displacement for the experimental data and FLAC 2D modelling simulation for double shear test without contact for indented SUMO cable bolt with 0 kN pretension load.
Figure 6-26 Comparison between the shear load versus shear displacement for the experimental data and FLAC 2D modelling simulation for double shear test without contact for plain SUMO cable bolt with 0 kN pretension load.

Figure 6-27 Comparison between the shear load versus shear displacement for the experimental data and FLAC 2D modelling simulation for double shear test without contact for plain MW10 cable bolt with 0 kN pretension load.
Figure 6- 28 Comparison between the shear load versus shear displacement for the experimental data and FLAC 2D modelling simulation for double shear test without contact for spiral MW9 cable bolt with 0 kN pretension load

6.4.2 Indented SUMO cable bolt subjected to British Standard test

The indented SUMO (ID-SUMO) cable bolt with 0 kN pretension load subjected to the British Standard test was simulated in this part. Elastic model was used for concrete blocks simulation similar to double shear tests. Joints were only created by the null function because there was no contact between concrete blocks and it was unnecessary to assign a model for the joints. The following material properties were used in the model as shown in Table 6-6. Modelling simulations and results are shown in Figures 6-29 to 6-32. Figure 6-29 shows the modelling simulation considering two concrete cylinders with 0.9 m length totally. Figure 6-30 shows the good agreement between experimental results and modelling simulation. Figure 6-31 shows the test results in 20, 40, 60 and 80 mm displacement. Figure 6-32 shows the shear stress contributions in the concrete blocks in British Standard test.
### Table 6- 4 Model properties of the indented SUMO cable bolt subjected to British Standard

<table>
<thead>
<tr>
<th>Steel materials</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk modulus</td>
<td>$160 \times 10^9$ Pa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>$79.3 \times 10^7$ Pa</td>
</tr>
<tr>
<td>Density</td>
<td>$7850 \text{ kg/m}^3$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cable bolt properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Area</td>
<td>$3.39 \times 10^{-4}$ m$^2$</td>
</tr>
<tr>
<td>Second moment of Inertia</td>
<td>$2.83 \times 10^{-6}$ m$^3$</td>
</tr>
<tr>
<td>Yield load</td>
<td>$568 \times 10^3$ N</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Grout properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear cohesion</td>
<td>$1.75 \times 10^5$ Pa</td>
</tr>
<tr>
<td>Shear stiffness</td>
<td>$1.12 \times 10^7$ Pa</td>
</tr>
<tr>
<td>Normal cohesion</td>
<td>$2 \times 10^6$ Pa</td>
</tr>
<tr>
<td>Normal stiffness</td>
<td>$1 \times 10^{10}$ Pa</td>
</tr>
</tbody>
</table>

**Figure 6- 29** Simulation of fully grouted cable bolt subjected to British standard
Figure 6- 30 Comparison between the shear load versus shear displacement for the experimental data and modelling simulation for British Standard test

Figure 6- 31 Shear load in the cable bolt for displacements of 20, 40, 60 and 80 mm for British Standard test. The x axis shows the shear displacement in mm, the y axis shows the shear load (The shear load unit is MN)
Figure 6-32 Shear stress contributions in the concrete blocks subjected to single shear tests (British Standard) without contact between concrete blocks in 100 mm displacement

6.4.3 Indented SUMO cable bolt subjected to single shear test

The indented SUMO (ID-SUMO) cable bolt with 0 kN pretension load subjected to single shear test is simulated in this part. Same simulation to double shear and British Standard tests were used for concrete cylinders and joint properties. The amount of normal stiffness was decreased compared with double shear testing because no barrel and wedge system was used for single shear tests. The following material properties were used in the model as shown in Table 6-7. Modelling simulations and results are summarised in Figures 6-33 to 6-35. Figure 6-33 shows the modelling simulation considering four concrete cylinders with 3.6 m length totally in single shear test. Figure 6-34 shows the good agreement between experimental results and modelling simulation. Figure 6-35 shows the shear stress contributions in the concrete blocks in British Standard test.
Table 6-5 Model properties of the indented SUMO cable bolt subjected to single shear test

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Concrete materials</strong></td>
<td></td>
</tr>
<tr>
<td>Bulk modulus</td>
<td>$23.56 \times 10^9$ Pa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>$16.94 \times 10^9$ Pa</td>
</tr>
<tr>
<td>Density</td>
<td>2400 kg/m$^3$</td>
</tr>
<tr>
<td><strong>Cable bolt properties</strong></td>
<td></td>
</tr>
<tr>
<td>Area</td>
<td>$3.39 \times 10^{-4}$ m$^2$</td>
</tr>
<tr>
<td>Second moment of Inertia</td>
<td>$2.83 \times 10^{-8}$ m$^4$</td>
</tr>
<tr>
<td>Yield load</td>
<td>$568 \times 10^3$ N</td>
</tr>
<tr>
<td><strong>Grout properties</strong></td>
<td></td>
</tr>
<tr>
<td>Shear cohesion</td>
<td>$1.75 \times 10^5$ Pa</td>
</tr>
<tr>
<td>Shear stiffness</td>
<td>$1.12 \times 10^7$ Pa</td>
</tr>
<tr>
<td>Normal cohesion</td>
<td>$1 \times 10^7$ Pa</td>
</tr>
<tr>
<td>Normal stiffness</td>
<td>$1 \times 10^8$ Pa</td>
</tr>
</tbody>
</table>

Figure 6-33 Simulation of fully grouted cable bolt subjected to single shear test
Figure 6-34 Comparison between the shear load versus shear displacement for the experimental data and modelling simulation for single shear test.

Figure 6-35 Shear stress contributions in the concrete blocks subjected to single shear tests in 100 mm displacement.
6. 5 Summary

The shear behaviour of pre-tensioned fully grouted cable bolts subjected to single and double shear tests and British Standards was simulated analytically and numerically. The performance of cable bolts subjected to double shear was simulated using energy balance theory and the Fourier series concept. Furthermore, the performance of cable bolts subjected to single shear test was simulated using the tri-linear method. Both models were capable of determining the performance of cable bolts in elastic, strain softening and failure stages. Both models were calibrated using the measured data for different types of cable bolts and pretension loads. Generally, the result of simulations using the constitutive model was in good agreement with the measured laboratory data observed for the shear and axial loads versus shear displacement.

Also, the performance of cable bolts was simulated using FLAC 2D. The properties of cable bolts, boundary conditions and loading were introduced precisely. The model was able to simulate the shear load versus shear displacement in different stages of loading. The results show that the FLAC 2D is capable of simulating the shear behaviour of cable bolts.
Chapter VII

FINAL REVIEW OF THE THESIS STUDY RESULTS

7.1 Introduction

The programme of shear testing of cable using single and double shear methods necessitated further examination and evaluation of tested shear strength values. Two series of double shear tests were conducted when (a) sheared concrete blocks were in contact with each other using MKII apparatus and (b) the newly modified double shear apparatus without concrete face contacts, thus eliminating the effect of friction using MKIII apparatus. Also, a series of single shear tests were conducted to study the performance of cable bolts in shearing. The main aim of this study was to evaluate the effects of various parameters including cable type, cable surface profile, birdcage and pretension load on the shear behaviour of cable bolts, which is achievable by comparing results from MKII, MKIII and single shear tests. Debonding characteristics of various cable bolts was only possible by the single shear method because of the length of cable bolt installation in concrete.

7.2 Comparison between the performance of cable bolts with and without contact between concrete block surfaces in double shear tests

The comparison between the performance of cable bolts in MKII and MKIII were studied in chapter IV. The high value of the peak shear load with contact faces was due to the combination of the shear loads due to cable bolt failure load plus the friction force resulting from rubbing concrete block faces in contact with each other. When concrete blocks were not in contact, the peak shear load decreased because it only included the
load due to the failure of the cable bolt. When the concrete blocks were not in contact with each other, the peak shear load was 54% of the peak shear load when the concrete blocks are in contact with each other. This difference in shear loading values clearly demonstrated the significance of the shearing force spent on overcoming the friction force between shearing concrete joint faces. Also the comparison between shear displacements at peak shear load showed that the shear displacement at peak shear load was higher when there was no contact between concrete blocks. This was due to higher freedom of movement of concrete blocks. However it is worth noting that two series of double shear tests were different in type of cable bolts and pretension load; therefore it would be useful to conduct more systematic tests to compare tests results to obtain more accurate results.

The limitations of double shear tests were:

- No debonding monitoring was possible in double shear tests due to the use of barrel and wedge fittings at the cable ends, constraining free movement of the cable ends
- The double shear assembly was unconfined due to the rectangular shape of the apparatus. Confinement impacts on the results as increased confinement will lead to improvement in bond strength.

These limitations in double shear testing methods were the reason to conduct single shear tests because the use of single shear apparatus allows longer cable encapsulation length and it was possible to monitor cable end movement/ debonding because of the unconstrained free ends of the cable protruding from the ends of long concrete samples.
7.3 Comparison between single shear and the double shear (MKIII) tests results

Table 7-1 and Figure 7-1 show differences in test performances between single and double shear tests. The shear displacement was greater in double shear test in comparison with single shear. This was attributed to the lack of effective confinement in the rectangular shape of concrete blocks in double shear test.

Table 7-1 and Figure 7-1 also show the result of similar cable types with similar pretension loads subjected to single and double shear tests. The double shear peak shear loads were divided by two to achieve the shear load on one face.

The following were observed from Table 7-1:

1. The peak shear load and shear displacement for plain SUMO cable were higher in single shear test compared with double shear test results. The cable in double shear tests with zero pretension load did not break, and the displacement at peak shear load was greater. The rate of loading was in general greater in single shear tests compared with consistent 1 mm/min rate at double shear test.

2. Generally, there were variations between the peak shear load between single and double shear failure loads and displacement at peak shear load. This was attributed to the lack of proper sample confinement in the double shear apparatus, thus no mechanism was in place to prevent the concrete from cracking during the shear stage. Very little cracking was observed with single shear test specimen because of the concrete cylinder confinement.
Chapter VIII

Conclusions and recommendations

Table 7-1 Comparison between the double and single shear tests results

<table>
<thead>
<tr>
<th>Cable type/pretension load (kN)</th>
<th>Double shear test (MKIII)</th>
<th>Single shear test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak shear load (kN)</td>
<td>Displ. at peak shear load (mm)</td>
</tr>
<tr>
<td>Plain SUMO/0</td>
<td>443*</td>
<td>100</td>
</tr>
<tr>
<td>Plain SUMO/150</td>
<td>426</td>
<td>88.2</td>
</tr>
<tr>
<td>ID SUMO/0</td>
<td>407.5</td>
<td>93.4</td>
</tr>
<tr>
<td>ID SUMO/150</td>
<td>383</td>
<td>85.7</td>
</tr>
<tr>
<td>UB Plain MW10/150</td>
<td>462</td>
<td>88.5</td>
</tr>
<tr>
<td>UB Spiral MW9/150</td>
<td>419</td>
<td>88.47</td>
</tr>
</tbody>
</table>

* Sample did not fail

Figure 7-1 Comparison between the peak shear load in MKIII and single shear tests
The amount of displacement at peak shear load in MKIII is higher than in the single shear test as shown in Figure 7-2. This is due to the use of barrel and wedge in double shear system, which increases the system stiffness. Thus, more displacement is required to achieve peak shear load compared with single shear test.

![Figure 7-2](image.png) Comparison of the displacement at peak shear load in MKIII and single shear tests

Advantages and disadvantages of both systems are shown in Table 7-2.
Table 7- 2 Advantages and disadvantages of single and double shear tests

<table>
<thead>
<tr>
<th>Method</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single shear</td>
<td>a. Results are realistic</td>
<td>a. Labour intensive</td>
</tr>
<tr>
<td></td>
<td>b. The technique allows monitoring cable debonding</td>
<td>b. Costly technique</td>
</tr>
<tr>
<td></td>
<td></td>
<td>c. Requires last space to undertake sample preparation and testing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>d. Difficult to control the rate of loading</td>
</tr>
<tr>
<td></td>
<td></td>
<td>e. Requires large laboratory space</td>
</tr>
<tr>
<td></td>
<td></td>
<td>f. Requires overhead heavy lifting gear</td>
</tr>
<tr>
<td>Double shear</td>
<td>a. Much simpler and faster method of testing</td>
<td>a. Samples are unconfined hence, greater shear displacement</td>
</tr>
<tr>
<td></td>
<td>b. Less expensive method</td>
<td>b. Unconfined samples crack, which may lead to greater shear displacement</td>
</tr>
<tr>
<td></td>
<td>c. Less space required for sample preparation</td>
<td>c. The technique does not allow monitoring cable debonding</td>
</tr>
<tr>
<td></td>
<td>d. Replicate field condition because there is more than one shear plane in</td>
<td></td>
</tr>
<tr>
<td></td>
<td>in situ</td>
<td></td>
</tr>
</tbody>
</table>

It is thus necessary to modify the double shear testing technique to facilitate preventing the concrete cracks that are not conducive to effective evaluation of the shearing performance of various cable bolts. Accordingly it is recommended that the double shear testing method be modified to enable applying effective confinement to the concrete blocks.
7.4 Development of a mathematical calculation of the effect of surface friction

A mathematical model was proposed using the energy balance theory and Fourier series concept to evaluate the performance of the pre-tensioned fully grouted cable bolts subjected to double shearing when the concrete blocks are in contact. This model consists of three stages of shear behaviour versus displacement as elastic, strain softening and failure. The cabled rock joint performance in the elastic and strain softening stages were proposed as linear. The most critical stage was the failure stage which was developed as:

\[ N = \frac{a_0}{2} + a_1 \cos\left(\frac{2\pi u}{T}\right) + a_2 \cos\left(\frac{4\pi u}{T}\right) + a_3 \cos\left(\frac{6\pi u}{T}\right) \]  

\[ S_1 = N (\cos \cot \theta - \sin \theta) + \sigma_y A_t \cot \theta + \sigma_s A_s \]  

where, \( N \) is axial load, \( S_1 \) is shear load, \( dv \) is shear displacement, \( du \) is axial displacement, \( \sigma_y \) is yield strength of cable bolt, \( \sigma_s \) is shear strength of cable bolt, \( A_s \) is the broken area of cable in shear, and \( A_t \) is the broken area of cable in tension, \( \cot \theta = \frac{du}{dv} \), \( a_n \) and \( b_n \) are Fourier coefficients, \( n \) is the number of Fourier coefficient, \( u \) is the shear displacement and \( T \) is the shearing length.

Therefore, the shear load in the failure stage was derived from Equations 7.1 and 7.2, as:

\[ S_1 = \left[ \frac{a_0}{2} + a_1 \cos\left(\frac{2\pi u}{T}\right) + a_2 \cos\left(\frac{4\pi u}{T}\right) + a_3 \cos\left(\frac{6\pi u}{T}\right) \right] (\cos \cot \theta - \sin \theta) + \sigma_y A_t \cot \theta + \sigma_s A_s \]  

This model could be extended to measure the performance of the pre-tensioned cable bolts when the concrete blocks were not in contact (MKIII). It is worth noting that
However the effect of friction on the shear strength of system in MKII was not considered directly but it was calculated in the axial load generated in the cable and also in the failure angle \( i \) of cable bolt. Thus, it is reasonable to deduct the amount of friction from equation 7.3 to measure the net shear strength of cable bolt in MKIII. The Mohr-Coulomb criterion measures the friction between concrete blocks with a simple equation:

\[
S_2 = N \tan \phi + c \tag{7.4}
\]

where, \( S_2 \) is shear load, \( N \) is axial load, \( \phi \) is friction angle between concrete blocks and \( c \) is cohesion.

For the current study, the shear load proposed from Equations 7.3 should be deducted from equation 7.4 to propose a model to evaluate the performance of the cabled jointed rock mass without contact between concrete blocks. Thus, the model is extended to:

\[
S = S_1 - S_2 \tag{7.5}
\]

Therefore;

\[
S = N(\cos i \cot i - \sin i) + \sigma_y A_t \cot i + \sigma_s A_s - N \tan \phi \tag{7.6}
\]

\[
S = N(\cos i \cot i - \sin i - \tan \phi) + \sigma_y A_t \cot i + \sigma_s A_s \tag{7.7}
\]

Therefore, the shear load in the failure stage is calculated as:

\[
S = [\sigma_0 + a_1 \cos \left(\frac{2\pi u}{T}\right) + a_2 \cos \left(\frac{4\pi u}{T}\right) + a_3 \cos \left(\frac{6\pi u}{T}\right)] (\cos i \cot i - \sin i - \tan \phi) + \sigma_y A_t \cot i + \sigma_s A_s \tag{7.8}
\]

The shear displacement at peak shear load is derived as:

\[
\frac{d(S)}{du} = 0 \tag{7.9}
\]
Therefore, the shear displacement at peak shear load is:

\[
    u_{\text{peak}} = \frac{T}{2\pi} \cos^{-1}\left( \frac{\sqrt{a_2^2 + 9a_3^2 - 3a_1a_3 - a_2}}{6a_3} \right)
\]  

(7.11)

Equation 7.12 shows the calculation to specify the peak shear load of the system when the concrete blocks are not in contact with each other. The peak shear load is calculated from equations 7.8 and 7.11:

\[
    S = \frac{a_0}{2} + a_1 \cos \left( \cos^{-1} \left( \frac{\sqrt{a_2^2 + 9a_3^2 - 3a_1a_3 - a_2}}{6a_3} \right) \right) + a_2 \cos \left( 2 \cos^{-1} \left( \frac{\sqrt{a_2^2 + 9a_3^2 - 3a_1a_3 - a_2}}{6a_3} \right) \right) + a_3 \cos \left( 3 \cos^{-1} \left( \frac{\sqrt{a_2^2 + 9a_3^2 - 3a_1a_3 - a_2}}{6a_3} \right) \right) (\cos i \cot i - \sin i - \tan \theta) + \sigma_yA_t \cot i + \sigma_sA_s
\]  

(7.12)

Using equation 7.12 for the plain 19 wire, 21.7 mm cable bolt with 10 t pretension load shows the accuracy of the current model. The peak load for the plain 19 wire, 21.7 mm cable without contact per surface is 369 kN. Table 7-3 shows the peak shear load measured from equation 7.12.

<table>
<thead>
<tr>
<th>Test</th>
<th>Test</th>
<th>(a_0)</th>
<th>(a_1)</th>
<th>(a_2)</th>
<th>(a_3)</th>
<th>Peak shear load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain 19 wire, 21.7 mm cable</td>
<td>637.03</td>
<td>-79.95</td>
<td>-3.38</td>
<td>-1.92</td>
<td>365.8</td>
<td></td>
</tr>
</tbody>
</table>

Table 7-3 Model simulation result for 19 wire, 21.7 mm cable bolt with 10 t pretension load
The peak shear load is measured as 365.8 kN for the model while the peak shear load for the test is 369 kN. This comparison shows the accuracy of the current model.

7.5 Summary

The results achieved from two series of double shear tests, MKII and MKIII, and single shear tests were compared in this chapter. The single shear apparatus allowed exploring the effect of debonding and confinement on the peak shear load which could not be discovered through the double shear test, MKIII.

- The result of MKIII on the same type of cable bolt and similar pretension load was about 55% of MKII which is due to removing contact between concrete block surfaces. This shows the high impact of concrete block surface resistance in shear strength of the system. However limited number of similar tests was conducted in MKII and MKIII. Thus, it is not reasonable to make a conclusion about the contribution of concrete blocks in total shear strength of the system in MKII.

- The result for rough surface profile cable bolts was less affected by the type of shearing apparatus because the results achieved from MKIII and single shear test were constant.

- The result of smooth surface cable bolts in double shear and single shear were varied because of debonding in the single shear tests. However, the double shear test has some advantages in comparison with single shear test including monitoring of axial load during pre-tensioning and testing and constant shear rate which was not present in the single shear test. On the other hand, single shear test has some benefits in monitoring bonding and debonding of cable bolts and using confined cylinders which replicates field conditions better. Thus, each technique
has its own advantages and disadvantages and their results together provide a comprehensive source of data to study the performance of cable bolts in shear.

➢ The proposed mathematical model is also capable of determining the peak shear load of the system accurately when there is no contact between concrete surfaces.
CONCLUSIONS AND RECOMMENDATIONS

Several aspects of the shear behaviour of cable bolts under shear and single shear tests were studied. The laboratory testing programme included:

- Assessing the shear behaviour of cable bolts under double shear tests when concrete blocks were in contact (MKII). Samples were cast using 40 MPa concrete blocks encapsulated by various types of cementitious grouts and chemical resins. 12 double shear tests were conducted on indented Superstrand, plain Superstrand, indented TG, indented SUMO, plain SUMO, Garford twin-strand, Secura HGC and plain 19 wires, 21.7 mm (9 x 9 x 1). The pretension load on the cable bolts were 0 t, 10 t and 25 t. The shear strength of the system was dependent on the cable bolt strength and friction between concrete block surfaces. The rating of shearing was constant at 1 mm/min.

- Assessing the shear behaviour of cable bolts under double shear tests when concrete blocks were not in contact (MKIII). Samples were cast using 40 MPa concrete blocks and Stratabinder HS grout. 12 double shear tests were conducted on plain 19 wires, 21.7 mm (9x9x1), plain SUMO, indented SUMO, un-bulbed spiral MW9 and un-bulbed plain MW10 cable bolts. The pretension load on the cable bolts were 0 and 15 t. The friction between concrete blocks was removed; therefore, the measured shear load is the shear strength of cable bolt. The rate of shearing was constant at 1 mm/min.

- Assessing the shear behaviour of cable bolts under single shear test. The Stratabinder HS grout and 40 MPa concrete cylinders were used. 16 single shear
tests on bulbed and un-bulbed spiral MW9, bulbed and un-bulbed plain MW10, Secura HGC, plain SUMO, indented SUMO, indented TG, plain Superstrand and plain Garford were conducted. The shear load was measured to determine the shear strength of cable bolts. The shearing was applied to the samples manually and thus the shear rates were inconstant. The samples were confined, which replicated the field condition.

The analytical and numerical simulations included:

- An incremental elasto-plastic constitutive model was proposed to predict the shear behaviour of cable bolt and friction between concrete block surfaces under double shear test (MKII) using Fourier series concept and energy balance theory. The model was calibrated using experimental results. This model was capable to determine the shear strength of the system in elastic, strain softening and failure stages.

- An analytical model was proposed using tri-linear method to simulate the shear performance of cable bolts subjected to single shear test. The model was calibrated using experimental results to predict the net strength of cable bolts in shear.

- FLAC 2D was used for simulation of cable bolts subjected to double shear (MKIII), British Standards and single shear tests.

**Cable bolt Research**

To date, most of the research is concerned with the experimental study of performance of rock bolts and cable bolts in pull testing to study the load transfer characteristics of rock bolts and cable bolts. Various analytical models and numerical simulations were proposed to estimate the tensile performance of cable bolts and rock bolts. Also, a
number of experimental, mathematical and numerical simulations have been conducted to study the shear performance of rock bolts. Some of the published research was intended to study the shear behaviour of cable bolts under single and double shear tests experimentally. The performance of cable bolts in shear under double shear test was mostly conducted at University of Wollongong. Only a limited research work was reported on the analytical models and numerical simulations to investigate the shear behaviour of cable bolts in different stages of loading.

**Shear behaviour of cable bolts in double shear testing (MKII and MKIII) and single shear testing**

- The shear behaviour of each cable bolt has three stages of elastic, strain softening and failure.
- Cable type, pretension load, failure angle, lay length and the concrete block strength were important factors in the peak shear load of the pre-tensioned fully grouted cable bolts.
- The contact between concrete block surfaces was on average between 70% and 90% of the total surface area in double shear testing with contact between concrete block surfaces (MKII).
- The cable deformation due to shearing was mostly between 60 and 80 mm from the contact surfaces when the concrete blocks were in contact with each other (MKII). This amount changed to 65 to 110 mm without contact between concrete block surfaces (MKIII). This was due to the higher freedom in the second series of double shear test.
The peak shear load of the plain cable was higher than the spiral one for the same type and pretension loads. The reason was possibly and primarily due to the reduction in the cross sectional area of the rough wire during indentation.

The ultimate tensile strength, lay length, number of wires, surface profile type (plain, spiral and indented) were important factors in the performance of cable bolts in shear. The Plain MW10 has the highest amount of peak shear load due to its increased number of wires in the strand to 10 wires, thus increased UTS, 600 mm lay length and plain surface profile type.

No debonding was occurred due to the use of barrel and wedge in the double shear assembly when tested.

The contribution of cable bolt to the total shear strength of the system was about 55%, while the contribution of concrete blocks to the total shear strength was about 45% in the double shear test with contact between concrete block surfaces (MKII).

The assessment of deboning characteristics of various cable bolts was not possible when tested in double shear. The extremity encapsulation length and the mounting of barrel and wedge assembly prevented deboning.

The embedment length (3.6 m) for plain cable bolts was not sufficient to prevent debonding in single shear test. Thus, additional embedment length is required to prevent debonding.

No debonding was observed in spiral and indented cable bolts in single shear test because of interlocking in the cable/grout interface.

Failure modes in the cable bolt were found to be a combination of both tensile and shear failure. Tensile failure occurs due to pulling cable bolts during shearing.
• The peak shear load and displacement at peak shear load decreased by increasing the pretension load because of increasing the system stiffness.

• The result of double shear tests with no friction was nearly compatible with single shear tests for rough surface cable bolts. There were some variances in the test results due to different test methodologies and shear rating.

• Confining of the concrete has attributed to maintaining concrete stiffness and preventing it cracking up during shearing process. Testing of cable bolt in rectangular blocks did not provide real confinement.

• The result of double shear tests with no friction was not compatible with single shear tests for smooth surface cable bolts due to debonding and high shear rate in single shear tests.

**Modelling of the behaviour of cable bolts in shear**

• The value of shear load in double shear testing methodology was predicted by the constitutive model, which was in good agreement with the experimental results.

• The additional shear resistance generated by the friction between concrete block surfaces was determined by the proposed model in elastic, strain softening and failure stages in double shearing (MKII).

• The FLAC 2D was used to simulate the performance of cable bolts in double shearing and British Standard, and single shearing methods. The FLAC 2D is capable in simulating the performance of cable bolts in shear precisely.
8.1 Recommendations for future research

The study of the performance of cable bolts in shearing can be extended to address the following recommendations, which have not been fully investigated within the scope of this research.

- **Concrete block strength**

  The rock mass was replaced by concrete blocks with 40 MPa strength in this study to keep consistency of all tests and to make comparison between different types of cable bolts. Using rock mass with similar properties is impossible however, rock mass in the field has different strength. Thus, it is recommended to conduct further tests using concrete blocks with different strength to provide comprehensive database in the cable bolts study.

- **Shear rate**

  The rate of shearing applied in the single shear tests was higher than 1mm/min. The shearing in the single shear test was applied manually varied between 1.37 mm/min and 38 mm/min. This high shear rate may affect the rationality of the results and affects the results obtained from single shear test. The extreme acceleration of the hydraulic pressure applies the shock loading to the sample and the results derived from single shear might be higher than the real values by removing the material creep from the results. Therefore, it is recommended to use a controlled application of load instead of the manual application of pressure for future studies.

- **Embedment length**

  Smooth surface profile cable bolts faced debonding in single shear tests because the cable embedment length (3.6 m) was less than the required amount. No debonding
was observed in the rough surface profile cable bolts due to interlocking between cable/grout interfaces. Thus, it is recommended to follow up a new series of single shear test with larger embedment length to overcome debonding and determine the shear strength of plain surface profile cable bolts.

• **Bulb structures**

The influence of bulbs was inconclusive because of the limited number of tests undertaken in single shear tests. More tests are required to be carried on bulbed cables to compare bulbed against un-bulbed cables.

• **Cable bolt’s installation angle**

The angle of cable bolts to the joints was kept constant at 90° for all single and double shear tests. However the cable installation angle is varied in in situ. Thus, it is desirable to design and conduct further tests on inclined cable bolts to determine the effect of cable bolt’s installation angle in shearing performance of the system.

• **Bonding and debonding in double shear test**

No confinement was provided in double shear assembly due to the concrete configuration. Thus, to represent the field condition, it is required to design a new double shear assembly (MKIV) with cylindrical concrete samples to provide confinement (Figure 8-1). Thus, it is recommended to design this new developed double shear assembly which is the topic of future research at University of Wollongong. This also provides a better comparison with single shear to specify the results accuracy.
Figure 8- 1 Double shear assembly with cylinder concrete samples


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APPENDICES A

SINGLE SHEAR TESTS RESULTS

Test 1: Un-bulbed plain MW10 cable bolt with 15 t pretension load
Appendices A

Single shear test results

Strain gauge 3 (percentage)

Shear displacement (mm)

Cable end movement (mm)

Shear displacement (mm)
Test 2: Bulbed plain MW10 cable bolt with 0 t pretension load

Strain gauge 1 (percentage)

Strain gauge 3 (percentage)
Test 3: Bulbed plain MW10 cable bolt with 15 t pretension load

Strain gauge 1 (percentage)

Strain gauge 3 (percentage)
Appendices A

Single shear test results

![Graph showing shear displacement vs. cable end movement.](image-url)
Test 4: Bulbed spiral MW9 cable bolt with 0 t pretension load

Strain gauge 1 (percentage) vs. Shear displacement (mm)

Strain gauge 2 (percentage) vs. Shear displacement (mm)
Test 5: Bulbed spiral MW9 cable bolt with 15 t pretension load

**Right end**

![Graph showing cable end displacement and shear displacement for the right end.]

**Left end**

![Graph showing cable end displacement and shear displacement for the left end.]

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Test 6: Un-bulbed spiral MW9 cable bolt with 15 t pretension load
Appendices A

Single shear test results

![Shear displacement vs. Strain gauge 3](image)

![Shear displacement vs. Strain gauge 4](image)
Test 7: Bulbed Secura HGC cable bolt with 0 t pretension load

LVDT right

LVDT left
Test 8: Bulbed Secura HGC cable bolt with 15 t pretension load

**LVDT right**

**LVDT left**
Appendices A

Single shear test results

Test 9: Bulbed plain SUMO cable bolt with 0 t pretension load

Right end

![Graph showing shear displacement vs. cable end displacement for the right end.]

Left end

![Graph showing shear displacement vs. cable end displacement for the left end.]

Shear displacement (mm)

Cable end displacement (mm)
Appendices A  

Single shear test results

Test 10: Bulbed plain SUMO cable bolt with 15 t pretension load

**Right end**

![Graph showing cable end displacement vs shear displacement for the right end.]

**Left end**

![Graph showing cable end displacement vs shear displacement for the left end.]

Test 11: Bulbed Indented SUMO cable bolt with 0 t pretension load

**LVDT right**

![Graph showing LVDT right displacement](image)

**LVDT left**

![Graph showing LVDT left displacement](image)
Test 12: Bulbed Indented SUMO cable bolt with 15 t pretension load

**LVDT right**

**LVDT left**
Test 13: Bulbed Indented TG cable bolt with 0 t pretension load

**Right end**

**Left end**
Test 14: Bulbed Indented TG cable bolt with 15 t pretension load

**Right end**

<table>
<thead>
<tr>
<th>Cable end displacement (mm)</th>
<th>Shear displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
</tr>
<tr>
<td>3.9</td>
<td>20</td>
</tr>
<tr>
<td>3.8</td>
<td>30</td>
</tr>
<tr>
<td>3.7</td>
<td>40</td>
</tr>
<tr>
<td>3.6</td>
<td>50</td>
</tr>
<tr>
<td>3.5</td>
<td>60</td>
</tr>
<tr>
<td>3.5</td>
<td>70</td>
</tr>
<tr>
<td>3.5</td>
<td>80</td>
</tr>
</tbody>
</table>

**Left end**

<table>
<thead>
<tr>
<th>Cable end displacement (mm)</th>
<th>Shear displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.2</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>8.8</td>
<td>20</td>
</tr>
<tr>
<td>8.4</td>
<td>30</td>
</tr>
<tr>
<td>8.2</td>
<td>40</td>
</tr>
<tr>
<td>8</td>
<td>50</td>
</tr>
<tr>
<td>7.8</td>
<td>60</td>
</tr>
<tr>
<td>7.8</td>
<td>70</td>
</tr>
<tr>
<td>7.8</td>
<td>80</td>
</tr>
</tbody>
</table>
Test 15: Un-bulbed plain Superstrand cable bolt with 0 t pretension load

Right end

Left end
Test 16: Un-bulbed plain Garford cable bolt with 0 t pretension load

[Graph showing cable end displacement vs. shear displacement for the right and left ends]
Test 17: Un-bulbed plain MW10 cable bolt with 15 t pretension load

**LVDT-Right**

**LVDT-Left**
Test 18: Bulbed plain MW10 cable bolt with 0 t pretension load

**LVDT - Right**

**LVDT - Left**
Test 19: Bulbed plain MW10 cable bolt with 15 t pretension load

**LVDT_RHS (mm)**

![Graph of cable end displacement vs. LVDT_RHS](image1)

**LVDT_LHS (mm)**

![Graph of cable end displacement vs. LVDT_LHS](image2)
APPENDICES B

FISH SUBROUTINE PROGRAM FOR THE NUMERICAL SIMULATION

OF THE CABLE BOLT

This part consists of following simulations:

1. Simulating the performance of plain Superstrand cable bolt subjected to double shear test,

2. Simulating the performance of indented SUMO cable bolt subjected to British Standard test, and

3. Simulating the performance of indented SUMO cable bolt subjected to single shear test.
1. Simulating the performance of plain Superstrand cable bolt subjected to double shear test

new

;Double shear without contact, plain Superstrand cable bolt

config

g 105 30

; --- concrete blocks properties ---

model elastic

prop bulk=23.56e9 shear=16.94e9

prop dens=2400

gen 0.0,0.0 0.0,0.3 1.05,0.3 1.05,0.0 i=1,106 j=1,31

; ---Joint ---

model null i=31, j=1,31

model null i=76, j=1,31

; ---Boundaries ---

pl hold grid

fix x y i=1

fix x y i=106

fix x y j=1, i=1,30
fix x y j=1, i=77,106

fix x y j=31, i=1,30

fix x y j=31, i=77,106

; --- cable bolt installation ---

stru rockbolt beg 0 0.15 end 1.05 0.15 seg 105 prop 4001

stru pro 4001 e 120e10 a 3.13e-4 cs_scoh 1.75e5 cs_sstiff 1.12e7 per 0.08

stru pro 4001 yield 525e5

stru pro 4001 i=1.1e-8 ; 0.25*pi*r^4

stru prop 4001 cs_nstiff 1e10 cs_ncoh 2e6 cs_nfric=45

stru prop 4001 pmom=2.25e4 tfs=1e-6

;Barrel and wedge

stru node 1 fix x

stru node 1 fix y

stru node 106 fix x

stru node 106 fix y

;Apply velocity

app yvel -1e-6 i=32,75 j=31

pl grid white rockbolt blu iface green

; --- Fish function to measure shear load
def ff

sum=0

loop i (1,31)

loop j (1,31)

sum=sum+yforce(i,j)

end_loop

end_loop

ff=sum

dd=step*1e-6

end

hist ff

hist dd

step 20000

pl hold struct shear

pl hold struct axial

pl hold hist -1 vs 2

pl hold syy fill

step 20000

pl hold struct shear
pl hold struct axial
pl hold hist -1 vs 2
pl hold syy fill
step 20000
pl hold struct shear
pl hold struct axial
pl hold hist -1 vs 2
pl hold syy fill
step 20000
pl hold struct shear
pl hold struct axial
pl hold hist -1 vs 2
pl hold syy fill
step 20000
pl hold struct shear
pl hold struct axial
pl hold hist -1 vs 2
pl hold syy fill
2. Simulating the performance of indented SUMO cable bolt subjected to 
British Standard test

new

; British standard test

config

g 90 70

; --- steel tube properties ---

model elastic

prop bulk=1.6e11 shear=79.3e9

prop dens=7850

gen 0.0,0.0 0.0,0.07 0.9,0.07 0.9,0.0 i=1,91 j=1,71

; ---Joint ---

model null i=46, j=1,71

; ---Boundaries ---

pl hold grid

fix x y i=1

fix x i=91

fix x y j=1, i=1,45

fix x y j=71, i=1,45
Appendices B

Single shear test results

fix x  j=1, i=47,91

fix x  j=71, i=47,91

; --- Cable bolt installation ---

stru rockbolt beg 0 0.035 end 0.9 0.035 seg 90 prop 4001

stru pro 4001 e 165e10 a 3.39e-4 cs_scoh 1.75e5 cs_sstiff 1.12e7 per 0.08

stru pro 4001 yield 568e5

stru pro 4001 i=2.82853e-8 ; 0.25*pi(r1^4-r2^4)

stru prop 4001 cs_nstiff 1e10 cs_ncoh 2e6 cs_nfric=45

stru prop 4001 pmom=2.25e4 tfs=3.6e-4

; Barrel and wedge

stru node 1 fix x

stru node 1 fix y

stru node 91 fix x

; Apply velocity

app  yvel -1e-6 i=47,91 j=71

; --- Fish function to measure shear load

def ff

sum=0

loop i (1,45)
Appendices B

Single shear test results

loop j (1,71)

sum=sum+yforce(i,j)

dd=step*1e-6

end

hist ff

hist dd

step 20000

pl hold struct shear

pl hold struct axial

pl hold hist -1 vs 2

pl hold syy fill

step 20000

pl hold struct shear

pl hold struct axial

pl hold hist -1 vs 2

pl hold syy fill
Appendices B

Single shear test results

step 20000

pl hold struct shear

pl hold struct axial

pl hold hist -1 vs 2

pl hold syy fill

step 20000

pl hold struct shear

pl hold struct axial

pl hold hist -1 vs 2

pl hold syy fill

step 20000

pl hold struct shear

pl hold struct axial

pl hold hist -1 vs 2

pl hold syy fill
3. Simulating the performance of indented SUMO cable bolt subjected to
   single shear test

new

; Single shear test, indented SUMO

config

g 360 25

; --- concrete properties ---

model elastic

prop bulk=23.56e9 shear=16.94e9

prop dens=2400

gen 0.0,0.0 0.0,0.25 3.6,0.25 3.6,0.0 i=1,361 j=1,26

; ---Joint ---

model null i=181, j=1,26

;---Boundaries ---

pl hold grid

;fix x y i=1

;fix x i=361

fix x y j=1, i=1,180

fix x y j=26, i=1,180

;fix x j=1, i=182,361

;fix x j=26, i=182,361

; --- Cable bolt installation ---

stru rockbolt beg 0 0.125 end 3.6 0.125 seg 90 prop 4001

stru pro 4001 e 95e8 a 3.3e-4 cs_scoh 1.75e5 cs_sstiff 1.12e7 per 0.08

stru pro 4001 yield 630e5
stru pro 4001 i=2.83e-8 ; 0.25*pi*r^4
stru prop 4001 cs_nstiff 1e8 cs_ncoh 1e7 cs_nfric=45
stru prop 4001 pmom=1.15e4 tfs=9.3e-2

;Apply velocity
app yvel -10e-6 i=182,361 j=26
plot hold grid white num iface blu

; --- Fish function to measure shear load

def ff
  sum=0
  loop i (1,181)
    loop j (1,26)
      sum=sum+yforce(i,j)
    end_loop
  end_loop
  ff=sum
  dd=step*10e-6
end
hist ff
hist dd
step 100000
pl hold struct shear
pl hold struct axial
pl hold hist -1 vs 2
pl hold syy fill
history write 1