Shear characterisation of various cable bolts using single and double shear techniques

Saman Khaleghparast

University of Wollongong

UNIVERSITY OF WOLLONGONG
COPYRIGHT WARNING

You may print or download ONE copy of this document for the purpose of your own research or study. The University does not authorise you to copy, communicate or otherwise make available electronically to any other person any copyright material contained on this site. You are reminded of the following:

This work is copyright. Apart from any use permitted under the Copyright Act 1968, no part of this work may be reproduced by any process, nor may any other exclusive right be exercised, without the permission of the author.

Copyright owners are entitled to take legal action against persons who infringe their copyright. A reproduction of material that is protected by copyright may be a copyright infringement. A court may impose penalties and award damages in relation to offences and infringements relating to copyright material. Higher penalties may apply, and higher damages may be awarded, for offences and infringements involving the conversion of material into digital or electronic form.

Unless otherwise indicated, the views expressed in this thesis are those of the author and do not necessarily represent the views of the University of Wollongong.

Recommended Citation
Shear characterisation of various cable bolts using single and double shear techniques

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

Master of Philosophy

from

UNIVERSITY OF WOLLONGONG

by

SAMAN KHALEGHPARAST

B.Eng (AUK), M.Eng (UOW)

Faculty of Engineering and Information Science

2017
CERTIFICATION

This research has been submitted in partial fulfilment of the requirements for the award of Master of Philosophy, in the school of Civil, Mining and Environmental Engineering, Faculty of Engineering and Information Sciences at the University of Wollongong.

Saman Khaleghparast

30 March 2017
ACKNOWLEDGMENT

I would like to express sincere appreciation to my thesis supervisor Professor Naj Aziz and Dr Ali Mirzaghorbanali and Associate Professor Shivakumar Karekal, Faculty of Engineering, University of Wollongong, who provided me the opportunity to conduct my thesis. Under your leadership, the University of Wollongong’s Rock Bolting Research Centre has become a global centre for optimising performance of ground support systems and it is my privilege to be a part of it.

Personally, I would like to further express my sincere appreciation to my parents and family for their encouragement to partake in all life has to offer. Special thanks to my father Alireza Khaleghparast and mother Mina Zangenehpour for their continuous support in all aspects of my life. Finally, I would like to dedicate this thesis to my father Alireza Khaleghparast, who has always been my support and inspiration, without his great impact, academia would not be present in my life.

I would also like to note a thank you to all the technical staff particularly Mr Cole Devonshire for your understanding and vast insight into all the practical aspects of my research. Furthermore, I would like to thank PhD candidates Haleh Rasekh, Guanyu Yang and Research assistant Xuwei Li for their support throughout the most tedious parts of testing and data analysis.
ABSTRACT

The application of cable bolts in Australian coal mines has been increasingly widespread over the past few decades. Cable bolts initially are installed as a secondary means of support systems in underground coal mines to complete the action of primary supports, rock bolts. Over the years, several investigations have been undertaken in order to better understand the mechanisms of rock and strata control to be able to more appropriately manage the environment of the ground. One of the most concerns amongst all researchers, which have been on increase, is to understand better the mechanism of load transferring systems in cable bolts. As a matter of fact, several investigations have been performed through utilizing Single and Double shear load test methodology to address the deficiency of previous studies and provide more credible and accurate database to be utilized by engineers and designers in order to create safer and more reliant environment in underground mining industry. The University of Wollongong in this regard has been playing a pivotal role to deal with the shortage of previous research studies in the field of Double and Single shear load tests and this thesis study continues the research work currently is going on at the research and development section of the University of Wollongong to enhance the scope of the previous studies.

In this experimental study two methodologies are adopted in order to examine the performance of most popular cable bolts using in Australian coal mines supplied by three dominant manufacturers, namely, Megabolt, Jennmar, and Minova. A new Single shear apparatus manufactured by Megabolt, is utilized to address the deficiencies of current single shear testing of cable bolts outlined in BS7861-2:2009. Accordingly, an eight different geometric cable bolts were examined through a sixteen set of tests by new single shear apparatus under various pretension values of zero and 15 tonnes to be able to investigate the effect of plain, spiral, bulb, no bulb and the combination of plain and indented wire strands with and without bulb configuration profiled cables on transferring load. Furthermore, a nine set of tests were conducted under zero and 15 tonnes pretension on four different geometric cable bolts through utilizing a new frictionless Double shear apparatus to explore the shear strength of such cables and understand the influence of plain and indented configurations on wire strands. The
results and data derived from each test were analysed and compared at the end before a reason conclusion is drawn.

The attained results from single shear tests clearly indicated that the plain profiled cable bolts in comparison with spiral cables having higher shear strength. Plain cable bolts underwent debonding during tests and this can evidence the fact that the embedment length of each side in single shear testing should not be considered as less than 1.8mm as this was insufficient for all plain profiled cables. In terms of bonding strength, the plain profiled cable bolts provided less bonding strength at the cable-grout interface compared to indented cable bolts and this was due to the surface roughness deficiencies of the plain wire strands. The effect of bulb profiled cable bolts was seen on plain cables to be more pronounced. On the other hand, the results achieved from double shear tests showed that the applied pretension has an adverse effect not only on the maximum shear load but corresponding displacement of the cables as well. It has also understood that the maximum shear strength of cable bolts are influenced by factors such as lay length, the surface profile of cables, the number of wires and the UTS. All tested cables demonstrated that the shear strength of each cable bolt was reached by approximately 65% of its UTS.
Table of Contents

CERTIFICATION.......................................................................................................................i
ACKNOWLEDGMENT..................................................................................................................ii
ABSTRACT.................................................................................................................................iii
Table of Contents.......................................................................................................................v
LIST OF FIGURES.......................................................................................................................viii
LIST OF TABLES..........................................................................................................................xiii
Nomenclature..............................................................................................................................xiv

Chapter 1: INTRODUCTION .................................................................................................. 15

1.1. General introduction ............................................................................................................. 15
1.2. Background for present research ....................................................................................... 17
1.3. Aim and objectives .............................................................................................................. 18
1.4. Research methodology ....................................................................................................... 18
1.5. Thesis outline ....................................................................................................................... 19

Chapter 2: GROUND SUPPORT MECHANISMS AND CABLE BOLTS FUNCTIONS .......... 20

2.1. Background of ground stresses and their redistribution with support .......................... 20
2.2. Rock bolts history ............................................................................................................. 22
2.3. Application of rock bolts .................................................................................................. 23
2.4. Bolt theories ....................................................................................................................... 23
2.5. Rock mass classification .................................................................................................... 25
2.6. History of the cable bolts and the evolving geometry configurations ...................... 28
2.7. Advantages of cable bolts ................................................................................................. 29
2.8. The cable bolt toolbox ....................................................................................................... 29
2.9. Cable bolt’s grout ............................................................................................................... 31
2.10. Workability of fresh cement paste .................................................................................. 31
2.11. Load transfer theory and associated failures ............................................................... 32

Chapter 3: THE LOAD TRANSFER ................................................................................... 36

3.1. Mechanism of cable bolts ................................................................................................. 36
3.2. Behaviour of reinforced rock ............................................................................................ 36
3.3. Load configurations for cable bolts .................................................................................. 37
3.4. Axial loading of cable bolts .............................................................................................. 38
3.5. Unconstrained single embedment pull test ..................................................................... 39
3.6. Rotationally constraint pull/push test .................................................. 41
3.6.1. Split-pipe pull/push test .................................................................. 41
3.6.2. Modified split-pipe pull/push test using Hoek cell ........................... 42
3.7. Double Embedment Pull Test (DEPT) ............................................... 44
3.8. Laboratory Short Encapsulation Pull Test (LSEPT) ............................ 46
3.9. Single shear test .................................................................................. 48
3.11. Double shear test ............................................................................. 51
3.12. Double shear testing of cable bolts .................................................. 56
3.13. Chapter summary ............................................................................ 64

Chapter 4: THE NEW SINGLE SHEAR TEST METHOD ....................... 65

4.1. Introduction ......................................................................................... 65
4.2. Megabolt single shear testing apparatus ........................................... 65
4.2.1. Components of the testing apparatus ............................................. 65
4.3. Experimental study parameters .......................................................... 67
4.3.1. The strength of the grout bonding agent ....................................... 69
4.3.2. Diameter of rifled holes ................................................................. 71
4.3.3. Pretension applied to the cable bolt .............................................. 71
4.3.4. Ultimate Compressive Strength of adjacent rock strata ............... 72
4.3.5. Embedment length ...................................................................... 73
4.3.6. Friction across the shear plane ..................................................... 73
4.4. Test sample preparation .................................................................... 74
4.4.1. Casting and curing of cylindrical concrete blocks ....................... 74
4.4.2. Adhesion of concrete anchor cylinder ......................................... 76
4.4.3. Cable bolt installation ................................................................. 80
4.4.4. Grouting the cable ...................................................................... 82
4.5. Testing procedure .............................................................................. 83
4.6. Chapter summary .............................................................................. 84

Chapter 5: DOUBLE SHEAR TEST METHOD ...................................... 85

5.1. Double shear testing apparatus .......................................................... 85
5.1.1. Component of the testing apparatus .......................................................... 85
5.2. Experimental study parameter .................................................................... 87
5.2.1. Type of cable bolt ...................................................................................... 87
5.2.2. Strength of the grout bonding agent ......................................................... 88
5.2.3. Diameter of rifled holes ........................................................................... 88
5.2.4. Pretension applied to the cable bolt ......................................................... 88
5.2.5. UCS adjacent rock mass ........................................................................... 89
5.3. Test sample preparation .............................................................................. 89
5.3.1. Unconfined Compressive Strength (UCS) test ........................................ 92
5.4. Cable bolt installation .................................................................................. 93
5.5. Testing procedure ....................................................................................... 94
5.6. Chapter summary ....................................................................................... 95

Chapter 6: RESULTS AND DISCUSSION ......................................................... 96

6.1. Test results ................................................................................................... 96
6.2. Load transfer performance of Plain MW10 cable bolt ................................ 98
6.3. Load transfer performance of Spiral MW9 cable bolt ................................. 103
6.4. Comparison of the plain and spiral cable configurations on the loading performance of cable .......................................................... 105
6.5. The effect of bulbed configurations on the loading performance of the cable ........................................................................................................ 107
6.6. Load transfer performance of Secura HGC cable bolt ............................... 108
6.7. Load transfer performance of Indented SUMO cable bolt ......................... 110
6.8. Load transfer performance of indented TG cable bolt ............................... 112
6.9. Load transfer performance of Plain SUMO cable bolt ............................... 114
6.10. Effect of indentation on the loading performance of cable ...................... 115
6.11. Load transfer performance of Superstrand cable bolt ............................ 116
6.12. Load transfer performance of Garford cable bolt .................................... 116
6.13. Double shear test analysis ......................................................................... 117
6.13.1. Load transfer performance of Plain SUMO ........................................... 117
6.13.2. Load transfer performance of Spiral SUMO .......................................... 119
6.13.3. Load transfer performance of Plain MW10 ........................................... 123
6.13.4. Load transfer performance of Spiral MW9 ......................................................... 126
6.15. Comparison of the plain and spiral cable geometry on the loading performance of cables ................................................................. 131
6.16. Reaction forces during the shearing ................................................................. 132
6.17. Cable bolt behaviour ......................................................................................... 132
   6.17.1. Elastic stage ................................................................................................. 132
   6.17.2. Plastic stage ................................................................................................. 133
   6.17.3. Axial load development ............................................................................. 133
6.18. Single and double shear test comparison ...................................................... 134
6.19. Chapter summary ............................................................................................. 140

Chapter 7: CONCLUSIONS AND RECOMMENDATIONS  .................. 141

REFERENCE LIST ........................................................................................................ 145
Appendix A ............................................................................................................... 152
Appendix B ............................................................................................................... 158
Appendix C ............................................................................................................... 160

LIST OF FIGURES

Figure 2-1: Ground reaction curve (adopted from (Thompson et al., 2012)) .......... 20
Figure 2-2: Conceptual ground response curve (adopted from (Galvin, 2016)) ....... 21
Figure 2-3: Typical roof support (adopted from (Goris et al., 1996)) ..................... 22
Figure 2-4: The application of rock bolts for ground stabilization in the world between 1930 and 1990 (adopted from (Jalalifar, 2006)) ................................. 23
Figure 2-5: Bolt theories (adopted from (Jalalifar, 2006)) ................................. 24
Figure 2-6: Element and array of cable bolts (Hutchinson and Diederichs, 1996) .... 28
Figure 2-7: Types of cable (adopted from (Hutchinson and Diederichs, 1996)) ..... 30
Figure 2-8: Schematic indication of the load transfer (adopted from (Thomas, 2012)) 32
Figure 2-9: Failure modes of load transfer related failure (adopted from (Hutchinson and Diederichs, 1996, Thomas, 2012)) ......................................................... 33
Figure 2-10: Detailed illustration of the load transfer concept (adopted from (Thomas, 2012)) ........................................................................................................ 34
Figure 3-1: Components of response for a fully grouted cable bolt with internal and external anchors (adopted from (Windsor, 1992)) ................................................................. 37
Figure 3-2: In situ loading of cable bolts (Hutchinson and Diederichs, 1996) ........... 38
Figure 3-3: The effect of pull out load for cables with rotation and non-rotation (adopted from (Hagan et al., 2015)) .................................................................................................................. 39
Figure 3-4: Unconstrained single embedment pull test example (adapted from (Brady and Brown, 2004)) ................................................................................................................................. 40
Figure 3-5: Split pipe pull test (adopted from (Fuller and Cox, 1975)) ................. 41
Figure 3-6: Pull test apparatus (Goris, 1990) ................................................................. 42
Figure 3-7: Modified Hoek Cell (MacSporran 1993) .................................................... 43
Figure 3-8: Load-Displacement graph for seven strand cable with varying confining pressures (MacSporran, 1993) ................................................................................................................................. 43
Figure 3-9: New modified Hoek Cells for modified-geometry cables ......................... 44
Figure 3-10: Double Embedment Pull Test example (Thomas, 2012) ....................... 45
Figure 3-11: Modified Laboratory Short Encapsulation Pull Test (Thomas, 2012) ...... 47
Figure 3-12: Modified LSEPT ACARP Project C22010 (Chen et al., 2014) ............. 48
Figure 3-13: Single shear apparatus utilizing two concrete blocks (Goris et al., 1996) . 49
Figure 3-14: The double embedment cable single shear testing frame (BSI 7861-2; 2009) .............................................................................................................................................. 50
Figure 3-15: General set up of MKI Sample in Instron (Aziz et al., 2003) ............... 53
Figure 3-16: Load-Displacement curves for 20MPa and 40MPa concrete strength (Aziz et al., 2003) ................................................................................................................................. 54
Figure 3-17: Experimental set-up for the angled testing of rockbolts with double bolted joints (Grasselli, 2005) ......................................................................................................................... 55
Figure 3-18: The effect of bolt orientation (Grasselli, 2005) ....................................... 55
Figure 3-19: Double shear box (Craig and Aziz, 2010a) ........................................... 57
Figure 3-20: Shear and axial Load-Displacement graph for the double shear testing of the Hollow TG cable (Craig and Aziz, 2010b) ................................................................. 58
Figure 3-21: General set up of MKII cable installation in concrete blocks (Aziz et al., 2014) ........................................................................................................................................ 58
Figure 3-22: Load-Displacement graph for Plain and Indented Hilti strand cable bolts (Aziz et al., 2014) ........................................................................................................................................................................59
Figure 3-23: Comparison of BS single shear and UOW double shear test for the Secura cable and plain and indented superstrand (Aziz et al., 2015b) ............................................................61
Figure 3-24: Painting the surface joint to show the friction (Rasekh et al., 2015) ....62
Figure 3-25: The opening of concrete blocks during test (Rasekh et al., 2015) ....62
Figure 3-26: MKIII- new frictionless double shear apparatus ..........................................63
Figure 4-1: Schematic of the Megabolt single shear apparatus (adopted from (Hawkins, 2016)) ............................................................................................................................................................................66
Figure 4-2: Single shear apparatus with a sample in position (adopted from (McKenzie and King, 2015) ) ........................................................................................................................................................................66
Figure 4-3: Apparatus pressure transducer and LVDT ....................................................67
Figure 4-4: Ultimate Compressive Strength of grout samples in relation to curing time (Mirza et al., 2016) ........................................................................................................................................................................70
Figure 4-5: Before and after ultimate compressive sample testing of sample 2 ..........71
Figure 4-6: Debonding of cable during loading (adopted from (McKenzie and King, 2015)) ........................................................................................................................................................................73
Figure 4-7: Applying the teflon film at the shear joint ....................................................74
Figure 4-8: Dimension of the cardboard cylinder moulds ..........................................75
Figure 4-9: Concrete casting and cardboard moulds ....................................................75
Figure 4-10: Schmidt Hammer .....................................................................................76
Figure 4-11: Base frame with clamped anchor cylinders .............................................77
Figure 4-12: Application of epoxy on cylinder joint surface .....................................77
Figure 4-13: Load bearing plate ..................................................................................78
Figure 4-14: Positioning of the anchor cylinder in the frame ....................................78
Figure 4-15: Application of washer seal and teflon plate at joint .........................79
Figure 4-16: Fixing the primary clamp to the shearing plane ...................................80
Figure 4-17: Strain gauge locations (adopted from (Hawkins, 2016)) ...................80
Figure 4-18: Application of strain gauge ..................................................................81
Figure 4-19: Pretensioning equipment and installation .................................................. 82
Figure 4-20: The installation of nut and spacer ............................................................ 82
Figure 4-21: Angled samples for grouting ................................................................. 83
Figure 4-22: LVDTs connected to the extremity of the cable ...................................... 84
Figure 5-1: Schematic of component the new MKIII .................................................. 85
Figure 5-2: Tensioned and grouted DST assembly ready for cap closing .................... 86
Figure 5-3: The schematic size of concrete blocks (adopted from (Rasekh et al., 2015)) .......................................................... 90
Figure 5-4: The prepared moulds as well as conduit and wire around it for riffling ...... 90
Figure 5-5: Sample procedure ..................................................................................... 91
Figure 5-6: Preparation of cylindrical samples ......................................................... 92
Figure 5-7: Process of UCS test ................................................................................. 92
Figure 5-8: Cable bolt installation and pretensioning process .................................... 94
Figure 5-9: View of double shear box testing ............................................................ 95
Figure 6-1: Plain MW10 test results ......................................................................... 99
Figure 6-2: Analysis of the failure mode of wires in Plain MW10.............................. 101
Figure 6-3: Test 1 - MW10 NB 15 tonne pretension - strand failure on the left anchor cylinder .................................................................................... 101
Figure 6-4: Test 1 strain gauge and LVDT measurements (Plain MW10 with no bulbs and 15 tonne pretension load) ................................................................. 102
Figure 6-5: Shear displacement corresponding to debonding at the cable-grout interface for MW10 samples .................................................................................... 103
Figure 6-6: Spiral MW9 test results ......................................................................... 104
Figure 6-7: Analysis of failure of strands in Spiral MW9 .......................................... 105
Figure 6-8: Comparison of the Plain and Spiral wire configuration ............................ 106
Figure 6-9: Effect of bulbed cable structure on the peak shear stress and displacement .................................................................................... 107
Figure 6-10: Secura HGC test results .................................................................... 108
Figure 6-11: Analysis of failure of strands of Test 7 and 8 (Secura HGC) ............. 109
LIST OF TABLES

Table 2-1: Classification of rock material based on unconfined compressive strength (adopted from (Goel and Singh, 2011)) .......................................................... 26
Table 2-2: Major rock mass classification systems (adopted from (Abbas and Konietzky, 2016)) .............................................................................................................. 27
Table 3-1: Experimental schedule indicating the number of samples test per bolt type (Aziz et al., 2003) .................................................................................................................. 52
Table 3-2: Yield point shear load values for different bolts under different environment (Aziz et al., 2003) .................................................................................................................. 53
Table 4-1: Types of cables tested ((Megabolt, 2016, Jennmar, 2016)) ....................... 68
Table 4-2: Stratabinder HS product specifications (Minova 2016) ............................ 69
Table 4-3: Stratabinder UCS test results (Orica, 2016) .............................................. 70
Table 4-4: Experimental plan of single shear testing of cable bolts ......................... 72
Table 4-5: Schmidt Hammer verification results ...................................................... 76
Table 4-6: Dimensions of teflon plate and washer seals ........................................... 79
Table 5-1: Type of cable bolt as tested in this study (Megabolt, 2016) ................. 87
Table 5-2: Single shear testing cables characteristics ............................................. 88
Table 6-1: Brief results of single shear test .............................................................. 96
Table 6-2: Brief results of double shear test ............................................................ 98
Table 6-3: Rate of loading ....................................................................................... 135
Table 6-4: Comparison of the single shear and double shear test results ............. 138
<table>
<thead>
<tr>
<th>Nomenclature</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACARP</td>
<td>Australian Coal Association Research Program</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
</tr>
<tr>
<td>BBBUGS</td>
<td>Bowen Basin Underground Geotechnical Society</td>
</tr>
<tr>
<td>BS</td>
<td>British Standards</td>
</tr>
<tr>
<td>DEPT</td>
<td>Double Embedment Pull Test</td>
</tr>
<tr>
<td>DST</td>
<td>Double Shear Testing</td>
</tr>
<tr>
<td>kN</td>
<td>kilo newton</td>
</tr>
<tr>
<td>LSEPT</td>
<td>Laboratory Short Encapsulation Pull Test</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Displacement Transducers</td>
</tr>
<tr>
<td>m</td>
<td>Metre</td>
</tr>
<tr>
<td>min</td>
<td>minute</td>
</tr>
<tr>
<td>mm</td>
<td>millimetre</td>
</tr>
<tr>
<td>mm/sec</td>
<td>Millimetre per second</td>
</tr>
<tr>
<td>mm/min</td>
<td>Millimetre per minute</td>
</tr>
<tr>
<td>MPa</td>
<td>Mega Pascals</td>
</tr>
<tr>
<td>t</td>
<td>tonnes</td>
</tr>
<tr>
<td>UCS</td>
<td>Uniaxial Compressive Strength</td>
</tr>
<tr>
<td>UOW</td>
<td>University of Wollongong</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>
1.1. General introduction

The instability of underground excavation as well as slopes is an important topic that attracts many researchers' attention over the past few decades, when particularly comes to engineering projects. Such issue has led to the occurrence of a considerable amount of fatal and non-fatal accidents in underground mining industry over the past few decades. According to Safe Work Australia, (2016) the number of decease was recorded as 10, 13 and 13 in 2013, 2014 and 2015 respectively. Consequently, one of the paramount concerns of researchers worldwide is the fact of dealing with the fatal and non-fatal accidents during and after excavation to attempt to reduce the number of deaths to zero. Therefore, it is significant for designers and engineers to broaden their knowledge to understand various forms of instability and the mechanisms of failure and related conditions in order to support the instable surfaces through utilizing and installing different types of rock bolts and cable bolts in more appropriate way.

To begin with, reinforcing the ground can be categorized into two distinct ways, primary and secondary reinforcement. Primary reinforcement refers to the immediate reinforcement of the ground layers during the excavation sequence through utilizing solid rock bolts. The principal purpose of using rock bolts is to decrease the rock mass deformation by increasing the linkage of the discontinued rock mass. Rock bolting is known as active support system as they apply forces to the rock mass to minimise the displacement of the jointed layers and loose rock units. In this case, the capability of tolerating stresses increase enormously. Rock bolting can be installed before and immediately after excavation in the mines. In order to have stable as well as reinforced structures, the needs of clearly understanding of the forms of instability, their mechanism, parameters and conditions is a must. Rock bolts are various and there are different effective factors to choose from, such as the number of bolts, the type of them as well as the regularity of their spaces, which are: rock conditions, aim of installation and lifespan of the excavated space (Bieniawski, 1984).

The secondary support system refers to the reinforcement applies sometime after the excavation. This is known as passive support system as they do not impose any initial force, instead they provide an extra resistive force to the medium to guarantee the
stability of the system when the rock deforms over time. Cable bolts are longer in length than rock bolts and act as a complimentary ground support system. The application of the secondary support system is when the bolted height does not provide adequate support to connect the discontinuities and fractured zone to a more stable and rigid strata layer. The usage of cables dates back to 1970’s and since then it has been a part of underground control support system in Australia (Hustrulid et al., 2001). Cable bolts are flexible tendons consists of multi-wire strand which has a high tensile strength and also are installed into drilled holes and bonded to the rock medium by grout. A very significant difference between cable bolts and other reinforcement elements is the bond length of them which is infinite (Windsor, 1992, Fuller, 1983, Hutchinson and Diederichs, 1996). The application of the cable bolts in today’s world as a supplementary underground support system has been on the increase for an integral excavation. According to Haleh et al. (2017), the installation of rock bolts worldwide were achieved to 500 million in 2011, and among this, 5,000,000, 600,000 and 250,000 are the share of Australia in utilizing bolts, cable bolts and split sets respectively.

The cable bolts, basically are required when the condition of the medium is one of the following (Hutchinson and Diederichs, 1996);

- Very weak roof areas.
- Faulted or broken ground.
- High ground stress areas.
- Large roadway spans.

The aim of cable bolting is to provide a more stabilised and strong rock mass by restricting the movement of instable and looser planes and instead, regulate post-failure deformation as well as providing resistance to bed separation (Galvin, 2016).

Underground support system has always been under investigation on the basis of two primary forces; axial loading and shear loading. Therefore, previous investigations were conducted in order to enable designers to selecting the most appropriate cable bolts based on the surrounding rock mass and geological environment for specific plane of weak layers (Thompson et al., 2012)
1.2. Background for present research

In the early 1900s, rock reinforcement application was initiated in the form of using timber. However, due to the fragile nature of timber which were reflect in a significant roof failure, brought so many researchers attention to address such disastrous issue. As a matter of fact, cable bolt was launched to the mining industry in the 1970s, primarily to surface mining and underground metalliferous mining, and it was in the 1980s that the application of cable bolts were introduced to coal mines and mostly for roadway reinforcement as a secondary means of support (Gerrard, 1983).

In the early 1950s the application of ground anchored for civil excavations commenced which was 60 years after the patent for pre-stressed concrete was presented. The usage of high tensile strength steel elements in the pre-stressed concrete industry was initiated in the mid-1960s. The earliest usage of cables was at the Willroy mine in Canada and also at the Free State Geduld Mines Ltd in South Africa. The principal reason for these first uses of cable bolts was to understand the possibility of supplying pre-stressed wire in long flexible length which would enable deep anchorage with rock mass without providing couple threaded of wires together (Windsor, 1992).

It was in the 1970s that the first pre-stressed cable bolt was made. It was consisted of seven wires, straight, 7mm diameter, high tensile strength and the steel wires were prepared with plastic spacer. The first type of cables which were introduced to mines as a temporary means of rock reinforcement was plain strand cable bolts. Due to smooth and straight configuration, such cable bolt was lack of load transfer capability. During the years, researchers started to work on the profile of the wires in order to improve the properties of the wires, in particular, load transfer capacity. Hence, a plain strand has been evolved in different configurations, such as strand surface profiling and indentation (Schmuck, 1979), double plain strand (Matthews et al., 1983), epoxy-coated strand (Dorsten et al., 1984), fiberglass cable bolt (Mah, 1994), the usage of birdcage in the strand (Hutchins et al., 1990), bulbed strand (Garford, 1990), nutcage strand cable bolts (Hyett, 1993).

The plain profile of cable bolts was turned into spiral profile for the first time and it was utilized in Broken Hill, Australia, in the early 1970’s. This transformation led to a great achievement in improving the adaptability, productivity, higher load transfer capability and mechanical performance. This type of cable bolts are strong and rigid enough to be
pushed into a 30 metre holes. Cables most commonly contain 7 wires weave together to shape a strong tendon (Hem, 2014). This type of cable with nominal diameter of 15.2mm is still the most prevalent ones to be utilized worldwide. According to Haleh et al. (2017), the majority of cables used in the USA and Canada are 15mm diameter with 7 strand which have the nominal strength of 27.22t. The central strand is called as “kingwire”, which is straight and the six others are slightly smaller bordering wires. The plain strand was modified by adding steel ferrules swaged at intervals on the strand (Windsor, 1992).

1.3. Aim and objectives

The aim of this research is to investigate the effect of various factors on the shear behaviour of a pre-tensioned fully grouted cable bolts utilized in Australia. These factors could be considered as: cable bolt type, amount of pre-tensioning and grout and rock properties. The principal objectives are:

- Study the previous research works in the field of the cable bolt behaviour subjected to two primary forces; axial and shear loading, in the form of critical literature review to investigate the load transfer capability of cable bolts in both tolerating and performing.
- Examine the effect of pre-tensioning on the shear strength and load transfer capacity of the cable bolt installed in rock formation.
- Evaluate the impact of different types of cable’s profiles (Spiral and Plain) on the shear strength of the reinforced rock with cable.

1.4. Research methodology

Two methods have been adopted for this experimental study of the shear strength of cable bolts, single shear and double shear methods in which the behaviour of shear strength of fully grouted cable bolts will be examined under different pre-tensioning. It is worth mentioning that the new single shear test apparatus is utilized to examine the performance of cables used in Australia in order to overcome the deficiencies of the current testing standard (Standard, 2009). Furthermore, double shear testing is conducted on different cable bolts through careful consideration of no shear face contact.
1.5. Thesis outline

This thesis comprises six chapters as follow:

Chapter 1 is an introduction and background of the research topic.

Chapter 2 presents the ground support mechanisms and cable bolts functions.

Chapter 3 is an in-depth literature review of the primary and fundamental studies in order to find performance details of cable bolts.

Chapter 4 covers the experimental study on the effective factors on shear strength of cable bolts subjected to single shearing test including introducing new design of the apparatus, sample preparation and testing procedure.

Chapter 5 presents the experimental study on the influential parameters on shear strength of cable bolts subjected to double shearing test including design of the apparatus, sample preparation and test procedure.

Chapter 6 demonstrates the results of both single and double shear test and discussion on the findings and a brief comparison between the two methodologies in order to examine the credibility and reliability of the results. It is followed by a conclusion and recommendation for future study in the topic.
2.1. Background of ground stresses and their redistribution with support

When an excavation comes to consideration it is commonly asserted that rock mass movement cannot be prevented (Thompson et al., 2012). The principal reason for rock mass movements, which in most cases would lead to failure, is the lack of confinement. Furthermore, pre-existing discontinued planes may cause failure due to unequal principal stresses caused by excavation in the newly formed free faces. It is understandable that the ground support must be provided in the vicinity of the excavation in order to decline or even eliminate the volume of failed materials as a result of rock mass movement. In order to do this, ground support system should provide adequate pressure to integrate and stabilize the surface to overcome or even minimize the deformation caused by excavation, in this sense, Thompson et al. (2012) provided “Ground Reaction Curve” to present such relationship.

![Ground Reaction Curve](image)

Figure 2-1: Ground reaction curve (adopted from (Thompson et al., 2012))
The characteristics of support systems are primarily featured by initial stiffness, load and yield capacity and in the case of roof to floor, stability (Galvin, 2016). The ground response curve illustrates in Figure 2-2 shows some of these elements. The basic concept derives from such concept is that once the interaction between ground response curve and support reaction curve occurs, no further convergence would take place, in other words, the equilibrium is attained, unless external situation affects the medium. It is worth mentioning that the stiffer support system, the swifter convergence takes place, this corresponds to the section labelled SE in Figure 2-2.

As a means of supporting system actively, rock bolts are applied immediately after excavation which, in turn, is called primary support system. The purpose of this primary support system is to connect the fractured bedding planes in strata overlaying to form a beam. Furthermore, the secondary support is installed in order to improve the endurance of the excavation. In this way, the aforementioned beam zone is connected to a more stable layer.

Figure 2-3 shows the schematic representation of primary and secondary support systems used in coal mines in Australia.
2.2. Rock bolts history

It was in the 1918 that the rock bolt was discovered to be used in coal mines by Germany. In 1872, the slate quarry in North Wales was the first in the United kingdom which reported the use of rock bolting in the coal mines (Schach et al., 1979). The use of mechanical rock bolt in a metal mine in 1927 in the United States was reported in 1983 by Bolstad and Hill. However, it was the Norwegian who developed the rock bolting as a practical and economical technology in the late 1940’s (Bolstad et al., 1983).

The major revolution in using roof bolting technology was begun in 1947 by the U.S Bureau of Mines (USBM) to decrease the number of fatal accidents caused by roof collapsing, and this was incrediblly spread throughout of the U.S till 1952. However, rock bolts application for the reinforcement and stabilization of underground coal mines is of worldwide scale nowadays, in which approximately 100 million roof bolts are installed every year in the U.S (Peng, 1985).

The rock bolting technology was begun to be used in Australia between 1949 and 1969 in the snowy mountain hydroelectric Scheme (Bolstad et al., 1983). Peng (1985) reported on the use of normal timber support roof bolting in Australia in Elrington Colliery, New South Wales. However, the annual amount of rock bolting used in underground opening mines in Australia was reported as an approximately 5 million.
2.3. Application of rock bolts

The rock bolt application, currently in principal of underground and ground surface reinforcement and stabilization is on the increase worldwide. Due to its popularity and the increase use of bolting system as a means of primary support system, there has been an increase trend in design. Basically, rock bolting can be utilized both as temporary and permanent support systems, in tunnelling and underground such as roadway development, shaft sinking operations as well as surface mining operations such as slope stability. Furthermore, rock bolts are applied to rock mass in order to prevent the movement of discontinued planes and poor bed separations.

According to Jalalifar (2006) it has been recorded that 15000 Km entries are excavated annually in the US coal mines in turn 100 million roof bolts are required to be installed in these entries. In a similar trend, in Australia, hundreds of millions of units are installed annually. Figure 2-4 clearly illustrates that the application of rock bolts for ground stabilization and reinforcement is an increasing trend in the world over in which Windsor (1997) reported that over 500 million rock bolts are installed every year worldwide.

![Figure 2-4: The application of rock bolts for ground stabilization in the world between 1930 and 1990 (adopted from (Jalalifar, 2006))](image)

2.4. Bolt theories

The action and reaction runs through the rock mass and the reinforcement support systems is complex and requires to be deeply understood what process is involved such as the rock mass configuration as well as the mechanisms of transferring load between
rock mass and the structural elements. Accordingly, any installed bolt is expected to be undergone pure axial or tensile, pure shear or combination of shear and tensile stress, when displacement occurs in the rock mass. Therefore, the behaviour of bolt requires close attention, in regard to above mentioned effective factors. However, there are various theories available for rock bolting as a ground support system, which are reliant on the methodology for the bolt application and the geological conditions (rock types). Figure 2-5 shows four different theories.

<table>
<thead>
<tr>
<th>Theory</th>
<th>Description</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suspension</td>
<td>The dead weight load of the strata transferred between the bolt head and the anchor.</td>
<td>The upper layer should be sufficient enough to anchor the bolts.</td>
</tr>
<tr>
<td>Beam building</td>
<td>Bolts bind the strata layers together, and increase its shear strength. The frictional effect generated by bolt pretensioning increases the shear strength between the layers.</td>
<td>The tensile failure is due to increase in binding force. However, failure may still occur at the ends. Hence bolts a and b are most effective in this method.</td>
</tr>
<tr>
<td>Keying effect</td>
<td>When the roof strata are highly fractured and weak, the immediate roof and weak strata, keying effect roof support may be used. Roof bolting provides significant real-time forces along the arches, cracks, and weak planes.</td>
<td>Confined by way of tensioned roof bolts, and then the strata material will be locked by key action.</td>
</tr>
<tr>
<td>Arching Action</td>
<td>The main aim of the arching theory is to increase the value of compressive stress in the roof so that the tensile stress is ignored and the shear stress provided by strong roof profile, which is one of the strongest rock profiles.</td>
<td>To support the weakened zones between bolts, metal mesh and shotcrete is recommended.</td>
</tr>
</tbody>
</table>

Figure 2-5: Bolt theories (adopted from Jalalifar, 2006)
2.5. Rock mass classification

The experimental design method on rock bolt is based on the rock mass classifications. Rock mass classification is a technical process to place rock mass into a certain groups or classes on defined relationships (Bieniawski, 1989, Abbas and Konietzky, 2016) to be able to predict the behaviour of the rock mass through the allocated number or given description. Moreover, rock mass classification enables designers as well as engineers to follow a guideline and optimize their engineering designs. A valuable systematic design aid has been provided through rock mass classification systems to many engineering projects in particular underground constructions, tunnelling and mining projects (Hoek, 2007).

In today’s world, the application of rock mass classification scheme has been increasingly prevalent among the researchers as it allows them to utilize such scheme in numerical simulations, especially when data are insufficient in the early stages of geotechnical projects. There are examples of utilizing such scheme subject to underground mining and slope stability presented by Chakraborti et al. (2012) and Herbst and Konietzky (2012).

An intact rock material behaves continuously, whilst a fractured is emerged in the rock mass and that would be conducive to discontinuities. This vicinity should be taken into consideration of any designer. However, the competency of rock mass is reliant on various factors to ensure rock mass stability. To begin with, the principal factor is the strength of the rock mass. This factor was proposed in ISO 14689-1 (2003) as it is shown in Table 2-1. The strength of the rock mass is possible to be measured by the uniaxial compressive strength in the field indirectly. The other important one is the Rock Quality Designtion (RQD). This is based on an improved core recovery technique, which is quantitative. This factor only includes sound pieces of core by length of 100 mm or greater. Another factor is discontinuities which are based on the spacing and condition (roughness, continuity, separation, joint-wall weathering, and infill), location of the discontinuity, groundwater conditions (inflow, pressure) and stress field.
Table 2-1: Classification of rock material based on unconfined compressive strength (adopted from (Goel and Singh, 2011))

In order to design underground structures various rock mass classification is adopted in which the well-known systems are listed in Table 2-2. It is worth noting that the most often usable schemes for assisting in designing underground structures are RMR, Q and GSI systems. (Goel and Singh, 2011, Abbas and Konietzky, 2016).
Table 2-2: Major rock mass classification systems (adopted from (Abbas and Konietzky, 2016))

<table>
<thead>
<tr>
<th>Rock Mass Classification System</th>
<th>Originator</th>
<th>Country of Origin</th>
<th>Application Areas</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock Load</td>
<td>Terzaghi, 1946</td>
<td>USA</td>
<td>Tunnels with steel Support</td>
</tr>
<tr>
<td>Stand-up time</td>
<td>Lauffer, 1958</td>
<td>Australia</td>
<td>Tunneling</td>
</tr>
<tr>
<td>New Austrian Tunneling Method (NATM)</td>
<td>Pacher et al., 1964</td>
<td>Austria</td>
<td>Tunneling</td>
</tr>
<tr>
<td>Rock Quality Designation (ROD)</td>
<td>Deere et al., 1967</td>
<td>USA</td>
<td>Core logging, tunneling</td>
</tr>
<tr>
<td>Rock Structure Rating (RSR)</td>
<td>Wickham et al., 1972</td>
<td>USA</td>
<td>Tunneling</td>
</tr>
<tr>
<td>Rock Mass Rating (RMR)</td>
<td>Bieniawski, 1973 (last modification 1989-USA)</td>
<td>South Africa</td>
<td>Tunnels, mines, (slopes, foundations)</td>
</tr>
<tr>
<td>Modified Rock Mass Rating (M-RMR)</td>
<td>Unal and Ozkan, 1990</td>
<td>Turkey</td>
<td>Mining</td>
</tr>
<tr>
<td>Rock Mass Quality (Q)</td>
<td>Barton et al., 1974 (last modification 2002)</td>
<td>Norway</td>
<td>Tunnels, mines, foundations</td>
</tr>
<tr>
<td>Strength-Block size</td>
<td>Franklin, 1975</td>
<td>Canada</td>
<td>Tunneling</td>
</tr>
<tr>
<td>Basic Geotechnical Classification</td>
<td>ISRM, 1981</td>
<td>International</td>
<td>General</td>
</tr>
<tr>
<td>Rock Mass Strength (RMS)</td>
<td>Stille et al., 1982</td>
<td>Sweden</td>
<td>Metal mining</td>
</tr>
<tr>
<td>Unified Rock Mass Classification System (URCS)</td>
<td>Williamson, 1984</td>
<td>USA</td>
<td>General</td>
</tr>
<tr>
<td>Communication Weakening Coefficient System (WCS)</td>
<td>Singh, 1986</td>
<td>India</td>
<td>Coal mining</td>
</tr>
<tr>
<td>Rock Mass Index (RMI)</td>
<td>Palmström, 1996</td>
<td>Sweden</td>
<td>Tunneling</td>
</tr>
<tr>
<td>Geological Strength Index (GSI)</td>
<td>Hoek and Brown, 1997</td>
<td>Canada</td>
<td>All underground excavations</td>
</tr>
</tbody>
</table>
2.6. **History of the cable bolts and the evolving geometry configurations**

Cable bolts is the main focus of this research study, as a matter of fact, cable bolts have been studied in more details.

One supporting system in rock bolting is cable bolting which is used in mining and civil engineering as a secondary support system. They can be used in different areas of roof and floor of underground and surface openings, walls, including: drifts and intersections, open stope backs, cut and fill stopes, open stope walls, draw points and permanent openings (Hutchinson and Diederichs, 1996, Windsor, 1992, Fuller, 1983). Cable bolts are flexible tendons consists of multi-wire strand which has a high tensile strength. A very significant difference between cable bolts and other reinforcement elements is the bond length of them which is infinite. Cable bolts length is usually considered between 10 and 25 meters with a load carrying capacity from 60 to 80 tonnes (Singh et al., 2016), however, it is possible to extend to 40 meters with advanced technology. The application of cable bolts is on the increase nowadays, in particular in the coal mines. The Figure 2-6 shows the array and element of cable bolts.

![Image of cable bolts installation](image)

*Figure 2-6: Element and array of cable bolts (Hutchinson and Diederichs, 1996)*

The installation of cable bolts is various. The first manner is to install the cable bolt in the hole, grout the anchorage at the far end, fit face restraint, pre-stress the cable bolts, and grout the balance of the hole. Another method is to pre-stress the cable bolt at the beginning and then grout the cable bolt. Pre or post-reinforcement is another significant characteristic of installing the cable bolt. In pre-reinforcement, the cable bolt preserves
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).

2.7. Advantages of cable bolts

Cable bolting support system is used in mining and civil engineering to:

- Provide a safe working environment,
- Stabilize rock mass,
- Control dilution of waste rock from the stope boundaries,
- Create installation of long bolts possible from limited working place,
- Arrange for a variety of performance characteristics by being invented using a number of different shapes of the steel wires,
- Place more than one cable bolt strand in a big single borehole to increase tensile capacity,
- Get used in a group with other support systems such as grouted rebar, mechanical bolts or shotcrete, and
- Make possibility to be restraint by attaching straps, plates and mesh.

2.8. The cable bolt toolbox

Plain strand cable bolt is the basic cable bolt that has been used worldwide for a number of years. However, in the last 20 years, different types of modified cable bolts strand have been developed to tackle the problems encountered with the poor performance of the plain strand cable bolts at mines. In fact, the cable bolt toolbox consists of variety of items to allow users as well as designers to have access to the truly effective cable bolts elements in order to prevent rock mass failure. The Figure 2-7 illustrates the different cable bolts configurations (Hutchinson and Diederichs, 1996).
<table>
<thead>
<tr>
<th>Types</th>
<th>Longitudinal Section</th>
<th>Cross Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single plain strand</td>
<td><img src="image1" alt="Image" /></td>
<td><img src="image2" alt="Image" /></td>
</tr>
<tr>
<td>Double plain strand with spacers</td>
<td><img src="image3" alt="Image" /></td>
<td><img src="image4" alt="Image" /></td>
</tr>
<tr>
<td>Birdcaged strand</td>
<td><img src="image5" alt="Image" /></td>
<td><img src="image6" alt="Image" /></td>
</tr>
<tr>
<td>Bulbed strand</td>
<td><img src="image7" alt="Image" /></td>
<td><img src="image8" alt="Image" /></td>
</tr>
<tr>
<td>Ferruled strand</td>
<td><img src="image9" alt="Image" /></td>
<td><img src="image10" alt="Image" /></td>
</tr>
<tr>
<td>Nutcaged strand</td>
<td><img src="image11" alt="Image" /></td>
<td><img src="image12" alt="Image" /></td>
</tr>
<tr>
<td>Epoxy-coated or encapsulated strand</td>
<td><img src="image13" alt="Image" /></td>
<td><img src="image14" alt="Image" /></td>
</tr>
<tr>
<td>Buttoned or swaged strand</td>
<td><img src="image15" alt="Image" /></td>
<td><img src="image16" alt="Image" /></td>
</tr>
</tbody>
</table>

Figure 2-7: Types of cable (adopted from (Hutchinson and Diederichs, 1996))
2.9. Cable bolt’s grout

Cable bolt, basically, is used to tolerate the extra stresses from discontinuities. Grout, therefore, is used to enable load to transfer effectively from rock mass to the cable bolt and vice versa. In other words, the duty of the grout is to provide a homogenous condition to enables the medium (rock mass and cable bolts) to work all together. The grout is normally a mixture of water and cement. In terms of the grout ratio (water: cement), the range of the ratio is differently observed at the mine sites from 0.3 to 0.6 or even 0.7 while this range of value should be between 0.3 to 0.4 for laboratory purposes. In order to increase its Unconfined Compressive Strength (UCS) the amount of water should be decreased, however, this can lead the process of pumping and grouting to become difficult since the less water, the thicker is the grout. Resin and shotcrete is used as of other types of grout. Fiberglass is also used as a substitute material for the cable bolts (Hutchinson and Diederichs, 1996).

2.10. Workability of fresh cement paste

Workability or in other words, the behaviour of grout during the process of placement is an important factor which needs to be considered. Initially, there are four principal concerns in regard to workability which are frequently contradictory (Hutchinson and Diederichs, 1996):

- The grout must have low viscosity or be fluid enough in order to allow it to be pumped some distance (usually 5m to 30m) along a tube of grout. In terms of its flowability as well as pumpability, the W: C ratio plays an important role in which by increasing the W: C ratio those factors are affected increasingly. However, W: C= 0.35 illustrates a lower limit for some commercial piston pumps whilst W: C= 0.3 has been successfully pumped and placed.
- The grout must be plastic enough or be viscous enough to hold itself in a borehole against the pull of gravity in a top-to-bottom installation. This can be met by decreasing the W: C ratio and the rate at less than 0.38 are recommended.
- The grout must be fluid enough to enter into the hollows of the steel strand cable or must fully penetrate the cage or bulb of modified strand cable, the cable therefore must fully encapsulate the cable.
The grout must resist primary set long enough to let pumping and placement. A grout of W: C= 0.3 will allow less than 15 minutes for placement.

2.11. **Load transfer theory and associated failures**

There are many factors affect choosing a more appropriate cable bolt as secondary support system such as cost, the convenience of installation, availability, capacity and grouting method. However, the principal element in which plays a pivotal role to failure or success of a system is the ability to transfer load from instable rock beds to stable rock mass through cable bolts. Even though all the aforementioned factors are required to be considered as effective factors to choose a proper cable bolts, the load transfer capability and capacity is still the significant ones and requires to be understood appropriately.

Transferring the load is carried out through a binding agent such as grout or resin, applied to the borehole covering around the cable bolts (Aziz and Jalalifar, 2005). Over the years, evolving the geometric structure of cable bolts has led to improvement the capability of cables to support loads, however, failure still occur and needs to be investigated in more details.

Initially, as Figure 2-8 indicates when a fracture takes place in a rock mass, simply the load is transferred from the unit below the fracture to the above unit of the fracture through the existence of mechanical interlock between cable bolts and the binding agent in the form of tensile load.

![Figure 2-8: Schematic indication of the load transfer (adopted from (Thomas, 2012))](image-url)
In fact, the five modes of possibility for grouted cable bolt to be failed presented by Hutchinson and Diederichs (1996), Thomas (2012) and Hagan et al. (2015), as follow (Figure 2-9):

A. By rupture of the steel tendon
B. At the cable/grout interface
C. Through the cable column
D. At the grout rock interface
E. Through the rock surrounding the borehole

![Figure 2-9: Failure modes of load transfer related failure (adopted from (Hutchinson and Diederichs, 1996, Thomas, 2012))](image)

Previous studies indicate that cables are unlikely to break and this is because the shear resistance between the cable bolts and the surface of the grout should be higher than the maximum tensile of the cable strand. However, the likelihood of failure in the cable-grout or even grout-rock interface is higher to occur since cable bolt and grout interface is a function of load transfer between the cable and rock unit (Chen et al., 2015). The most common modes of load transfer failure system amongst all the studies is the failure at the cable-grout interface (B) that is evaluated by dilation and this is because of having smaller contact area between cable-grout with comparison to the grout-rock.
Stillborg (1984) indicated that the resistance of a fully grouted cable bolt in a borehole through a block of concrete related to pull-out test, is provided by the following three mechanisms:

1. Chemical adhesion between the steel and the cement grout;
2. Friction, related to physical interlock between steel wire and the grout, and
3. Interlock, between the cable or steel wires and the grout.

The second and third mechanisms are more significant than the first one and in most cases the chemical adhesion mechanism is disregarded, particularly in analytical theories (Chen et al., 2014). As soon as the chemical adhesion is destroyed along the part of the cable, the bond between the cable and the grout is maintained by the friction as well as interlock. However, this bond can be faded as load increases so with a fraction slip at the interface, the cable starts to detach from the grout. By increasing the load continuously the detachment increases over the length of the cable. This is because of local crushing as well as micro-cracking in which finally de-bonding occurs through the cable.

![Diagram](image)

Figure 2-10: Detailed illustration of the load transfer concept (adopted from (Thomas, 2012))

Once the slippage occurred, the geometry of the cable starts deforming and that affects the grout immensely. The grout is extremely compressed and will ultimately expand, in other words more dilation force would create and pushes against the borehole face, and the wall of the borehole reacts and pushes back the pressure to the grout. This would
result in an increasing confinement on the cable and grout interface. The increased confinement pressure as a result of dilation would cause the shear strength of the cable and grout interface to increase and would end up leading to transferring load as cable is locked.

It is important to note that the bond between the cable and the grout is reliant on several factors such as type of cable (number and diameter of wires, properties of wires, indented or smooth, laying technique), the cable’s stiffness and diameter, the strength of the grout, borehole diameter, the embedment length and position of the cable in the borehole.

According to Galvin (2016), the performance of cable bolts is influenced by the following variables:

- Rock mass strength properties;
- Structural fabric of individual rock layers;
- in situ stress system;
- the type of cable bolts, length and density;
- the material properties of the cable bolt;
- the anchorage mechanism of the cable bolts;
- the direction of the stress;
- Rock mass failure mode; and
- The elapsed time of installation.

Moreover the shear performance of cable bolts is affected by many factors, including:

- Strength of the grout as bonding agent;
- The diameter of the borehole;
- The amount of the pretension applied to the cable bolt;
- Embedment length;
- The existence of friction across the shear interface; and
- Rock mass strength properties.
Chapter 3: THE LOAD TRANSFER

3.1. Mechanism of cable bolts

In this chapter, all major developments of the design of experimental testing methodologies with regard to the load transfer of cable bolts as well as different geometric structures are summarised. Both direct shear testing methods as well as pull-test methods will be investigated through this chapter, as axial load and shear load are two principal effective loads in this topic.

3.2. Behaviour of reinforced rock

Rock mass behaviour under reinforcement is complex and is controlled by the interface between grout and cable bolt. There are more additional important factors such as face restraint anchors and internal anchors in the behaviour of cable bolts as shown in Figure 3-1. This figure illustrates how displacement of the rock mass is variable on the basis of the interactions between all the mechanisms of the system. The cable bolt has an inherent twisting action which is getting coupled with a torsional interaction between outside surface of the peripheral wires and the grout. Therefore, the strand is getting pull out during deformation because of the shearing of the asperities and the torsion by rotating the strand. When the axial load increases, the high axial load combines with torsional force cause shear through peripheral wires and grout. It is expected that the failure surface has minor frictional response with mechanical interactions over an elemental length of strand. The ‘detachment or de-bonding front’ is defined as the travel of shear and torsion components from the point of load application. The grout strength controls the propagation of this front. Friction and radial stress control the behaviour of failure surface behind the propagating front. The steel surface roughness and the grout particle size affect the friction. Increasing the grout particle size provides higher coefficient of friction while it decreases grout strength and reduces the strength of interface. Moreover, it modifies the radial stress created during dilation. During the cable bolt installation and grout curing, a radial stress is sat up, which is affected during loading by different factors such as: composite Poisson’s ratio of the stand, torsion of cable, compaction and dilation of failed grout at the interface, and changes in the rock mass stress over the time (Windsor, 1992).
When the loading at a fully bonded strand is at discrete intervals along the length at intersections with discontinuities, the loading and displacement are more complex. It is possible for maximum six displacements in each discontinuity, three rotational and three translations. Also, the load can be a combination of axial, shear, bending and torsion. Crushing of the grout is another factor (Windsor, 1992).

The mechanism of transferring load in a fully grouted rock bolt has been studied by Jalalifar (2006), which is reliant on the shear stress over the surfaces of bolt-grout and grout-rock.

3.3. **Load configurations for cable bolts**

Naturally, rock mass contains various modes of discontinuities such as slips, fault, joints, and bedding planes, whilst during excavation new discontinuities propagate through rock mass. Consequently, rock mass reacts subjecting to these discontinuities and would tend to displace and deform, in this regard the role of bolting as well as cable bolting is emphasised as they have to stitch all these discontinuities together to prevent failure. In fact, the way the cable bolts as well as rock bolts are oriented is significantly important, some may be activated and start working once the rock mass attempts to fail. There are three modes of cable bolt loading including axial or tensile load, shear load and a combination of tensile and shear load. These types of cable bolt loading can occur individually or in combination within an array of cable bolts as depicted below in Figure 3-2 (Hutchinson and Diederichs, 1996).
3.4. **Axial loading of cable bolts**

Designing axial load test is relatively simple and it is possible to be conducted in the field and in the laboratory. Due to this availability and simplicity, there are huge numbers of data available regarding the performance of cable bolts in tension. It is worth mentioning that most of these tests that have been evolved and advanced over subsequent years to examine the axial load performance of cable bolts are obtained from the standard “Split pipe” test developed by Fuller and Cox (1975).

There are, basically, two types of tests to gauge the axial tension load, namely, an unconstraint test and non-rotating and double pipe test. Unconstraint test refers to a test in which the cable bolt allows to rotate which gives lower bond strength and it is simpler with comparison to non-rotating test. However, the other test is non-rotating and double pipe tests, in which the results overestimate the bond strength due to preventing the rotation of the cable. The lateral test is more complicated than the former test. The results of both these tests have been depicted in the form of load versus displacement graph as it is shown in Figure 3-3.

![Diagram of Axial and Shear Loading](image)

Figure 3-2: In situ loading of cable bolts (Hutchinson and Diederichs, 1996)
These two tests are controlled by different parameters in order to compare different grout and cable configurations, which are (Hutchinson and Diederichs, 1996):

- Type of cable bolt,
- Grout properties, such as: ratio of W: C, curing time, UCS,
- Length of embedment such as: type of test (anchor length or free pull length),
- Pipe dimension and material, which are borehole diameter and field test properties,
- Rate of pull,
- Load and displacement during pulling test, and
- Response of cable bolt, such as rotation, stick slip and cable strand rupture

3.5. **Unconstrained single embedment pull test**

This method of pull testing is the simplest to evaluate the performance of cable bolts under an axial load. In such a test, one leg of the cable is grouted into a rigid pipe and the other leg is free for the tensile testing machine to grip (Maloney et al., 1992, Chen et al., 2015).
Hyett and Bawden (1996), conducted single embedment pull out test to explore the effects of bulb frequency on the ultimate pull out capacity of Garford cable bolts and make a comparison between the results of bulbed profile cable bolts and results from previous plain strand cable bolts. The grout had 0.4 W: C ratio, which cured for 26 days and the embedment length, was varied from the range of 600, 900, and 1800mm.

Chen and Mitri (2005), and Aoki et al. (2002) performed similar test as above. The results of all these tests indicated that the bulb structure significantly increased the strength of the cable against the applied axial load. Also Chen et al. (2015) concluded that the axial strength of a bulbed strand cable increased by 35% in comparison with a plain strand cable.

This method was utilised by Mosse-Robinson and Sharrock (2010) to investigate the effect of borehole diameter on different bulb densities and grout quality. It was found that there was no relationship between the borehole diameter and the load capacity. This result was supported by Rajaie (1990).

Using single embedment pull testing method was prevalent among researchers on prematurely fundamental studies. However, unconstraint test allows cable to rotate resulting into lower bond strength when resisting the pulling load (Bawden et al., 1992) as shown in Figure 3-3. Chen et al. (2015), claimed that as the open part of the cable is not embedded it may tend to snap early, which is due to minimal torsional rigidity as well as its geometric structure. This method is utilized for primary testing of new cable variations as it is inexpensive (Tadolini et al., 2012), so it is more an in depth data required by industry to rely on.
3.6. Rotationally constraint pull/push test

3.6.1. Split-pipe pull/push test

Fuller and Cox (1975) developed constrained axial tests on cable bolts. In terms of their design, rock mass was represented by two mild steel tubes to confine the grouted cable bolt and a washer was utilized in order to represent the rock joint as it is shown in Figure 3-5. In this design because of the fact that the length of the support section was considered longer than that in embedment section, the lower part would always be in charge of failure.

![Figure 3-5: Split pipe pull test (adopted from (Fuller and Cox, 1975))](image)

The basics of the pioneer study of split pipe pull out test developed by Fuller and Cox (1975) was applied in new investigation by (Goris, 1990). The conducted study was similar with single embedment pull testing but the difference was the rotation of the other leg of cable, which was prevented by confining the sample under constant radial stress environment. The study compared single and double conventional, epoxy-coated, conventional with steel button and birdcage cables. Two steel pipes were used in the test which was connected by a rubber washer as shown in Figure 3-6. Cement grout with 0.45 W: C ratio was used and the test was carried out after 28 days of curing time. The axial load at the rate of 15mm/min to a total displacement of 150mm was provided by a 180 tonne capacity hydraulic pump. At the head of the machine a linear variable displacement transformers (LVDT) were used along with potentiometer on the cable sample in order to determine the loads as well as displacement during the test. This
study indicated that the strength of the cable with two strand increased by two-fold compare to cables tested.

Bawden et al. (1992) proposed a modification to this method. This study was conducted in order to compare the results of laboratory tests to field tests. The previous investigations indicated that the field test results with host strata of limestone/ash and granite in the Kingstone area, Canada, were significantly lower than the laboratory results due to the fact that In the laboratory the rock mass was stimulated by steel pipe (as per Goris 1990), however, in the alternated test modified by Bawden et al. (1992) in order to simulate the in-situ environment more appropriately, PVC as well as aluminium pipes were utilized as a means of inconstant in situ confinement environments because of the lower radial stiffness properties of such materials than steel.

3.6.2. Modified split-pipe pull/push test using Hoek cell
A changing to the typical pull test was proposed by MacSporran (1993). The modification was to confine the embedded part with a modified Hoek cell. The aim of using Hoek cell was to make sure that the confining pressure applied on the sample during the test was constant. This was problematic in the previous investigations.
MacSporran used the Hoek cell as shown in Figure 3-7 to compare results of different confining pressure of 2, 5, 10 and 15MPa applied to a seven strand cables. The load-displacement graph in Figure 3-8 illustrate that as the sample confinement pressure increased, the resisting maximum axial load increased accordingly. This result is in agreement with Bawden et al. (1992). However, this study remained uncertain about the improvement of bond strength with confinement.

Further investigation to clarify the relationship between the bond strength and confining pressure was undertaken by Hyett et al. (1995) on typical seven strand cable bolts with the use of Hoek Cell. Furthermore, this study was performed on different geometrical cable structures, in particular the Garford bulb by (Moosavi et al., 2002) as it is shown
in Figure 3-9. The initial finding of such studies was that the bond strength is a function of friction rather than adhesion in the cable-grout interface. Due to poor quality of grout, mining induced stress and minimal borehole radial stiffness, debonding is possible to occur with minimal radial dilations. The findings indicated that the higher radial dilation can be provided by modified cable geometries. It is also asserted that the generated axial load as well as radial dilation in the modified cables is higher than in the conventional cables due to the presence of bulbed structure.

Another study in this topic was conducted in University of Wollongong by Smith (2002). A Hoek triaxial test was used in such study to demonstrate the effect of changing in confining pressure on the rock bolt surface associated with resin. This study showed that these changes directly affected the shear performance of rock bolts. The experimental study indicated that as confinement pressure increases on the sample the shear resistance of the system increases accordingly. In addition, the higher confinement illustrated better performance under continuous loading in comparison with the lower confined samples.

3.7. Double Embedment Pull Test (DEPT)

The double embedment pull test was developed by Hutchins, et al. (1990) to evaluate the performance of birdcage structure with the impact of bulb location on the pull-out load. This new methodology enabled researchers to investigate the effect of embedment
lengths on both sides of the discontinuities. In this method, cable is installed into two steel tubes with encapsulation of grout or resin. An adaptor is installed at the end of the apparatus in each head to allow the tensile force to be applied, show in Figure 3-10.

By using this new methodology, the investigation asserts that the birdcage structure can increase the system resistance enormously under pull-out load. This was claimed by comparing the birdcage structure strands to typical plain strands. The reason for this improved load transfer was because of the increase in cross-sectional area of the cable strand, which would ultimately lead to improving the strand-grout interface area. Surprisingly, the study also investigated that pull-out load of the system in the case of typical cable bolt was reduced by the impact of debonding with painting the strand surface, whilst the birdcage structured cable bolts indicated conversely as it was not observed any reduction in load performance. Furthermore, Renwick (1992) undertook this methodology on Ultra-strand in order to compare the birdcage cable structure to plain cable structure, however, its result was not convincing. Moreover, it is worth pointing out the drawback of such method as this methodology is unable to assess the behaviour of grout-rock interface due to the steel tube. This was conducive to the creation of the Laboratory Short Encapsulation Test (LSET).
3.8. **Laboratory Short Encapsulation Pull Test (LSEPT)**

The DEPT methodology created by Hutchinson, et al. (1990) was adopted by Clifford, et al. (2001) in order to assess the behaviour of grout-rock interface. This was done through encapsulating a full cable into a genuine sandstone core sample along with rifling to better indicate the underground excavation. This new methodology (LSEPT) enables researchers to find out more comprehensive factors of failure mechanisms, including radial stiffness and bond strength. This methodology was the point of starting the effect of pretention on the load transfer mechanism to be studied as a result of the effectiveness of double embedment. The experimental plan of such method (LSEPT) was also adopted in British Standard (BS 7861-2, 2007) which is discussed further in Section 3.10.

A modified LSEPT apparatus was planned to conduct pull test on 14 cables types presently used in Australian coal mines by Thomas (2012). This study as shown in Figure 3-11 was conducted on 142mm sandstone core with grouted cable which has different UCS between 19MPa and 25MPa. Different diameter was allocated to the borehole with regard to cable bolts in use. The hydraulic ram was installed in order to provide axial load at the rate of 10KN/s.
The aim of this test which was conducted by Thomas (2012) was to provide a more realistic condition of in situ underground environment through modifying the method from Clifford et al (2001), which was achieved by:

- The thick-walled steel cylinder was replaced on the sample in order to replicate the highly variable applied stress in the field and not to fail under constraint rotation, which the biaxial cell was not capable of providing this.
- In order to prevent the unwinding of the cable, an anti-rotation device was applied to the sample, which was problematic in the previous studies.
- Grout and breather tubes were fixed

This study supported that bulbed or nutcaged structure of cable bolts enable the system to resist higher elastic load in comparison with typical plain strand. This experimental study demonstrated that not only the maximum stiffness of the cable fluctuated between two and three fold and also its capacity increased by 400% through varying cable structures. However, the results were affected by the increasing diameter of the borehole, which completely influenced bulbed and nutcage but reduced plain strand, impacting the reliability of the results.
The methodology provided by Thomas (2012) was modified further by Chen et al. (2014), in the ACARP project CC22010 by using concrete cylinders to simulate the rock mass, as shown in Figure 3-12. In this method by using split cylinders which were bolted on the sample, the application of the confining pressure was simulated.

![Diagram of Modified LSEPT ACARP Project C22010](image)

Figure 3-12: Modified LSEPT ACARP Project C22010 (Chen et al., 2014)

This study initially was conducted to extend the scope of the effect of both the sample geometry on the pull-out load as well as the roughness of borehole on the strand wire’s performance. It was claimed that concrete sample with a diameter of less than 300mm can transfer the pull load with minimal changes in the cable strand while concrete with wider diameter incapable of doing this. In addition, it was summarized that the cable tested can be affected by the borehole roughness, the plain wire cable indicated that the failure occurred at the cable-grout interface was due to debonding and neither smooth nor rifled borehole did not affect the failure significantly. On the other hand, data presented that the modified cable geometries can be influenced by the roughness of the borehole and the smooth borehole is more likely to affect the failure at the grout-rock interface, which did not take place with the rifled borehole.

3.9. Single shear test

A shear testing program was developed by Goris (1996), which used two 0.025m$^3$ concrete blocks with joint surfaces of smooth to rough tested. The concrete, which contained a fine sand-concrete mix with a 28-day compressive strength of 69MPa, was
poured into steel moulds. The joint surface was made through an aluminium-cast joint surface print to ensure integrity across the numerous samples. The characteristics of the two joints:

- Smooth – Joint Roughness Coefficient (JRC) 2 and 30° of angle friction
- Rough – JRC of 12 and 41.6° of angle friction

The cable was installed across both blocks and grouted as shown in Figure 3-13 to ensure full contact between the cable and the two concrete blocks and shear boxes.

![Figure 3-13: Single shear apparatus utilizing two concrete blocks (Goris et al., 1996)](image)

The study found that a created joint through concrete blocks can increase the shear resistance in cable by more than two times in both surfaces tested, smooth and rough. The assumption for this study was that the anchorage of the cable was considered mechanically (Barrel and wedge) on either leg, however this is not indicative of practical applications where the cable has a mechanical anchor at one end and a faceplate and nut at the other. The lack of tightening mechanism prevents the impact of pretension to be investigated, whilst the test was emphasized the significance of grouting through making a comparison between ungrouted cables and grouted cable.
elements in which the former had a significantly lower maximum stress. Furthermore, similar tests were conducted by Wittenberg and Studney (2004), Wullenweber and Wittenberg (2005) and Mahony and Hagan (2006) to reinforce the findings of Goris et al. (1996) in both rebars and cable bolts.

3.10. **British Standard (BS7861-2:2009)**

The British Standard in specific code of practice BS7861-2 carried out the LSEPT methodology suggested by Clifford et al. (2001) to launch a new standard for single shear test to be utilized in industry. In fact, double embedment of cable bolt is recommended by British Standard when the ultimate shear strength is aimed to determine.

The testing samples are set up in two thick walled 125mm length hollow steel tubes with the following features:

- Internal diameter 5mm larger than the cable diameter;
- Wall thickness 50% of the cable diameter; and
- An internal surface patterned with a thread of 1mm deep by 2mm in pitch

The testing cable with a length of 250mm is inserted by hand centrally in a temporarily tube and then cover it with a low set resin. The covered sample with resin must be allowed to cure for no less than 24 hours prior to testing. During testing, the applied load must not exceed 10(N/mm²)/s until the peak is reached.

![Figure 3-14: The double embedment cable single shear testing frame (BSI 7861-2; 2009)](image_url)
The Guillotine style apparatus holds the cable in steel tubes throughout the test which reflects the result of the cable counterpart in shearing to be real. However, this practice is not indicative of the real condition of cable bolts in underground mining industry since cable bolts in real condition undergo both shear and tension because of the crushing of host material and adjacent to shear plane because rock naturally undergoes the brittle failure. In this practice the adjacent host material is steel, therefore, once the shear load is applied due to the ductility of the host material, does not actually deform. Moreover, the deficiency of this method in both encapsulation as well as incapability of pretensioning the cables affect directly to the shear strength of the cable bolts and ultimately would be underestimated.

3.11. **Double shear test**

Double shear test is another suitable methodology to evaluate the performance of bolt behaviour in rock mass reinforcement. Accordingly, the Swedish Rock Mechanic Research Foundation is the first place where conducts shear testing in hard rock reinforced by rock bolts, in 1974. A huge number of investigations have been carried out over the past few decades to explore the effect of different factors which are capable of influencing shear strength, such as the length and diameter of the bolt, number of bolts, the inclination of the bolts, the relative displacements in joints, joint roughness, the impact of compression, relative strength of rock and grout and elastic modulus of rock and grout.

The earliest traceable shear testing of rock bolts is reported by Bjurstrom (1974). This research indicated that there is a lack of understanding of considering the influence of the capacity of bolted joints to transfer shear forces when the stability as well as deformational behaviour of bolted jointed rock mass is under consideration. As a matter of fact, this research was conducted by specially shear tests on fully cement-bonded rock bolts encapsulated in blocks of granite. In this research the four following aspects of the bolt effects were considered:

- Tension force in the bolt
- Friction at the shear surface as a result of increased normal stress
- No pretension effect of the bolts, and
- Bolt inclination with respect to shear surface
It is concluded that the stiffening the shear and an increase of the shear strength at smaller displacement is as a result of inclining the bolt.

Another experiment performed by Haas (1976) subjected to shear testing using different types of bolts and anchors in blocks of limestone and shale. This explored that shear stress resistance can be increased to approximately 3.7 times when fully grouted bolts were utilized in order to secure natural fracture.

A study completed by Aziz, et al. (2003) performed double shear test on bolts. The aim of this study was to understand the impact of different axial loading conditions as well as different bolt surface profiles in the behaviour of reinforced bolts by running double shearing test on the fully grouted and axially tensioned bolts. In this study, totally 28 tests were undertaken on three different types of bolts that commonly use in Australia. These bolts have different configurations as well as load transfer characteristics. Two different types of concrete blocks with different strength of 40MPa and 20MPa were casted in three pieces of steel moulds in order to stimulate two different rocks with different strength. The size of the concrete blocks in the corners was 150x150x150mm$^3$ while the size of the block on the centre was 300x150x150mm$^3$.

Table 3-1 depicts the number of samples test per bolt type and the results are drawn in the Table 3-2.

Table 3-1: Experimental schedule indicating the number of samples test per bolt type (Aziz et al., 2003)

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>20MPa Concrete</th>
<th>40MPa Concrete</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20kN</td>
<td>50kN</td>
<td>80kN</td>
</tr>
<tr>
<td>AX</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>AXR</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>JAB</td>
<td>1</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Total</td>
<td>5</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>
Table 3-2: Yield point shear load values for different bolts under different environment (Aziz et al., 2003)

<table>
<thead>
<tr>
<th>Concrete strength (MPa)</th>
<th>Tensile load (kN)</th>
<th>Type 1 Shear Load at y/point (kN)</th>
<th>Type 1 Shear displace. at y/point (mm)</th>
<th>Type 2 Load at Y/point (kN)</th>
<th>Type 2 Shear displace. at y/point (mm)</th>
<th>Type 3 Load at Y/point (kN)</th>
<th>Type 3 Shear displace. at y/point (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>20</td>
<td>180</td>
<td>2.79</td>
<td>240</td>
<td>5.84</td>
<td>240</td>
<td>3.88</td>
</tr>
<tr>
<td>40</td>
<td>50</td>
<td>200</td>
<td>4.59</td>
<td>240</td>
<td>4.86</td>
<td>300</td>
<td>5.21</td>
</tr>
<tr>
<td>40</td>
<td>80</td>
<td>240</td>
<td>3.37</td>
<td>240</td>
<td>4.38</td>
<td>180</td>
<td>3.58</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
<td>100</td>
<td>4.89</td>
<td>160</td>
<td>8.86</td>
<td>80</td>
<td>6.51</td>
</tr>
<tr>
<td>20</td>
<td>50</td>
<td>150</td>
<td>5.86</td>
<td>160</td>
<td>4.64</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>20</td>
<td>80</td>
<td>180</td>
<td>5.84</td>
<td>160</td>
<td>4.69</td>
<td>180</td>
<td>4.61</td>
</tr>
</tbody>
</table>

The results indicated that there was no constant pattern for behaviour of bolts, however, by increasing the initial axial load, the shear load increases chiefly. This pattern was more constant for concrete blocks with higher strength compared to the weaker ones. This study shows that the bolt type two had a constant shear load at all three stage of the bolt tension loads in both 20MPa and 40MPa concrete medium. The holes for allocating the bolt in the concrete blocks were not rifled; it was a good reason to consider the related movement between grout-concrete and bolt.

Another study completed by Aziz, et al. (2003) planned a three-part concrete block double shear apparatus to stimulate shear performance of rebar under different axial loading conditions.

Figure 3-15: General set up of MKI Sample in Instron (Aziz et al., 2003)
Two different strength, 20 and 40MPa, concrete blocks were casted in this set of experiment in wooden moulds with central dimension of 600×150×150mm$^3$ double jointed with two cubic concrete blocks of 150×150×150mm$^3$. A plastic conduit was placed across the centre face of the concrete blocks to provide a hole to place a rebar after curing. In 30 days when the concrete cured, the central hole was reamed out and the rebar was fully encapsulated through the implementation of grout and pretensioning. Unlike the BSI (2009), this practice created a full encapsulation of cable bolt inside of the concrete blocks which stimulate experienced in-situ conditions closely.

The data derived from this practical study suggested that the strength of the host material has a direct correlation with the maximum shear load of the system as the higher strength concrete, would experience a higher peak shear load, this trend can be seen from the above Figure 3-16. Furthermore, the load transfer mechanism was influenced by applied pretensioning load, this was proven by comparison the results from the various applied pretension load. The testing methodology due to its relatively small scale and the limited capacity of the Instron Universal Testing Machine is insufficient for cable bolts which require increased size applied loads and volume of host material.

Simultaneously a larger scale development in the double shear test was carried out by Grasselli (2005) to investigate the shear performance of solid bolts that commonly utilized in road tunnel. The testing assemblage includes three blocks of concrete with a dimension of 1000× 600×600mm$^3$. In this set up the bolt was installed in different angle to investigate the effectiveness of orientation of the bolt on its performance as shown in Figure 3-17.
Fully cement grouted 16mm and 20mm diameter bolts and Swellex dowel were utilized through this experimental study. The procedure of applying shear load in this study was similar to what Aziz et al (2003) did in their experiment through pushing the central block with the assistance of a downward motion. The surface of all samples was macroscopically smooth to eliminate uncertainty across tests as reported by Goris (1996).

Grasselli reported that as the bolt orientation increases from the horizon to 45°, the maximum shear load increased for 20mm rebar as reported in above Figure 3-18. Moreover, the findings showed that a bolt set at an angle of 45° experienced smaller displacement and could resist a higher peak load in comparison with a bolt set at an

![Figure 3-17: Experimental set-up for the angled testing of rockbolts with double bolted joints (Grasselli, 2005)](image)

![Figure 3-18: The effect of bolt orientation (Grasselli, 2005)](image)
angle of zero degree or completely horizon. In this practical test methodology confining pressure was disregarded and that can affect not only the accuracy of the test but also the simulation of the field conditions as the lack of confining pressure is conducive to reducing the influence of excessive load resultant from the additional stress from the weight of overlaying rock, in this case, the host cementitious material. Therefore, because of this deficiency, aforementioned method cannot be accepted to be utilized for shear testing of cables, unless the effect of confining pressure to be applied.

It is worth pointing out to an earlier investigation on single shear test conducted by Haas (1976) as the host materials were adopted was chalk and limestone to investigate the shear performance of bolts in different orientation as well as considering the pressures applied to the shear interface. The study was summarized that the shear displacement was reduced due to the increase of the pressure in the fractured zone by utilizing rock bolts. A greater peak shear load was determined through installing rock bolts at 45 degree in the shear face. Even though the testing assembly affect the equality of the load distribution to the shear face, the results verify the findings of Grasselli (2005).

3.12. Double shear testing of cable bolts

The performance of cable bolts in double shear test was performed by Craig and Aziz (2010b), which was the first solid study in this area. The method was utilized in this study was similar to that used by Aziz, et al. (2003), however the difference was the scale of the apparatus which was increased in order to examine the behaviour of pre-tensioned 28mm hollow strand TG cable bolt. The aim of this study was to evaluate the performance of the cable bolt in shear under variable axial load conditions and assess the failure mechanisms of the cable. The maximum amounts of machine traveling for the test were fixed to 50mm and 75mm.
As it is shown in Figure 3-19 three concrete blocks of 50MPa strength with a central dimension of 450×300×300mm$^3$ and two cubic concretes with a dimension of 300×300×300mm$^3$ was cast. A 42mm in diameter plastic conduit was inserted through the centre face of the concrete block to provide the rifled borehole to allow placing the cable for encapsulation. All cables were pretensioned prior to encapsulation process. Grout was pumped across the borehole through two central vertical cavities on the top face of the concrete blocks. The axial load was monitored through placing load cells on each extremity of the cables during the pretensioning process as well as the actual test.

The results of this experimental study claimed that the failure of a cable bolt in direct shear load is a combination failure of cable strands. During the applied shear load the cable bolt bends and that would result in concrete crushing at the shear face and then the shear load transfer across the concrete as well as cable bolts until it reaches the ultimate tensile strength of the steel wires. Point A indicates the first adjustment of the medium and the commencement of crushing of the host concrete and that would be reflect in the further bending in the cable at point B (Figure 3-20). This deformation is indicative of the less structural integrity between concrete host material and cable bolts which would ultimately result in failure, this can be proved by the reduce gradient after point B in Figure 3-20.
The double shear test outlined in Craig and Aziz (2010b) was utilized by Aziz, et al. (2014) to report the shear performance of various cable geometries. The aim of this study was to investigate the shear performance of two Hilti-Plain and spiral Strand cable bolt configurations. A modification was carried out to the apparatus in which the vertical hole placed on the top of each block was removed and the grouting process was conducted through pumping of grout from one leg of the cable to another to be able to simulate closely the in situ environment as it is shown in Figure 3-21. The cable bolt was fixed into the sample by using barrel and wedge on either extremity and in this way, the applied pretensioned was also maintained. The sample was loaded by a 500t compression testing machine with a greater vertical displacement was used to allow for the cable to fully shear, which did not occur in the previous study.

Figure 3-20: Shear and axial Load-Displacement graph for the double shear testing of the Hollow TG cable (Craig and Aziz, 2010b)

Figure 3-21: General set up of MKII cable installation in concrete blocks (Aziz et al., 2014)
A constant shear load at 1mm/min for all samples in accordance with the BS7861-2:2009 was exerted. The load-displacement graphs for both the spiral and plain strand geometries were illustrated at Figure 3-22. It can be seen from the graphs that the plain wire configuration was not completely failed, however the peak shear stress reached in the plain wire surpassed the peak shear load of the indented Hilti. As it can be seen from the graphs the plain cable bolt did not fail completely, whilst the Hilti cable bolt underwent complete failure and the peak shear load reached by Plain wire configurations is relatively higher than that achieved by Hilti wire strands. This study suggested that the surface geometry of each wire can be influenced the shear performance of a cable. This was proved by the indented sample as the achieved peak shear load was 13.8% lower than that reached by the plain strand because of the influence of the indentation on the structural integrity of the wires. Furthermore this study, in relation to numerical modelling by Jalalifar, et al. (2006) showed that data collect from using the British Standard (BS7861-2:2009) may be misleading as discussed in Section 3.9.

Figure 3-22: Load-Displacement graph for Plain and Indented Hilti strand cable bolts (Aziz et al., 2014)
Further investigation on double shear test was carried out by Aziz et al. (2015a). In this study the both plain and indented wire configurations of SUMO and TG along with the Garford twin strand were utilized to investigate their loading performance under different applied pretension load as well as birdcage structure. The results indicated that the birdcage structure had a detrimental effect on the shear loading performance of the indented strand. Moreover, this study confirmed the result given by Aziz et al. (2014) that the indentation reduces the shear strength compared to the plain configuration of wire strands. The study reported that there is a direct correlation between Ultimate tensile strength ad shear strength of cables. The analysis of the results of the study generated a mathematical model to predict shear strength which is discussed further in this chapter.

Simultaneously, the spiral and plain profile strands was studied to explore the shear strength performance (Aziz, et al. 2015), as well as the Secura Hollow which is a combination of smooth and indented wires (Aziz, et al. 2015) at the University of Wollongong through double shear test. The study by Aziz, et al. (2015) compared the results from plain strands to the indented strands and concluded that the shear loading of the medium is reduced by 13.8% in the spirally profile strands. It was asserted that the lesser cross-sectional area was the principal reason for such reduction in shear strength performance in spirally indented wires configuration. Both tests conducted reach an identical conclusion in regards to the inconsistencies of the British Standard (BS7861-2:2009) which completely underestimated the strength of the cable as shown in Figure 3-23. The studies summarized that the incapability of imposing a pre-tension load on the sample and the guillotine effect induced by the apparatus in association with the steel tubes imposed pure shear on the sample, which is not reflective of the standards followed in the University of Wollongong double shear test dictated by in-situ failure features.
Rasekh, et al., (2015) conducted double shear test to evaluate the effect of concrete friction in order to calculate the shear strength of the cable bolt. It is believed that shear force components are quantified and it is possible to be determined. The applied shear force is consumed in:

- Shearing and bending of the cable bolt,
- Overcoming the shearing friction of the two concrete joint force, and
- Shearing of the grout annulus which is small and can be considered as part of the concrete joint.

The test was exerted a similar method to that used by Craig and Aziz (2010). The strength of the concrete was considered as 40Mpa. Since this study conducted to evaluate the effect of joint friction on shearing, the surface of the concrete blocks (joints) was painted using special pattern shown in Figure 3-24. Each surface was divided to 36 squares in order to allow measuring of the contact of the concretes at the end of the test by counting the damaged squares.
In this study, first double shear test had conducted on three blocks of concrete without instalment of cable bolt in order to evaluate the concrete blocks sliding properties and then 10 different types of cable bolt with different geometry (spiral and smooth) as well as different pre-tensioned load was tested. The Mohr Coulomb equation was developed in this study to measure the friction angle, which was considered as 26.94°. The study found that the contact between the shear joints is not 100%, this is because when load is applied to the centre block, a gap opens between the concrete blocks as it is shown in Figure 3-25 and it can be considered as 70% to 85%.

Aziz, et al., (2015) claimed that 10% of the shear load is assigned to the rubbing of the concrete surfaces and consequently 90% of the evaluated shear load integrated in calculating the shear stress value.

Recently, a modification has been carried out to the University of Wollongong double shear test apparatus and that has created the third generation of previous double shear apparatus. In this version, two horizontal braces are placed in each side of the sample
specimen in order to prevent friction between the jointed concrete faces as it is shown in Figure 3-26. This newly modified apparatus has reflected in the shear results as the friction of the shear faces was eliminated and that resulted in pure shear strength for cables. It was reported that 30% of the shear loads from the previous UOW studies were originated from the friction at the joints (Aziz et al., 2016a).

Figure 3-26: MKIII- new frictionless double shear apparatus
3.13. **Chapter summary**

Generally, majority of scientific findings in terms of the loading performance throughout the increasing of this secondary support has been on the basis of pull testing as it has its own positive ramifications such as being simple in design, relatively inexpensive and can be conducted in the laboratory or even in the field. It has been so many innovation and creativity carrying out in principal of pull-out test apparatus over the past five decades and in this regard a recently publication of a new-rotating pull test apparatus by (Aziz et al., 2015c), has caused more quality and dependable data to be created for pull out load transfer mechanism for cables, however, it is worth mentioning that this does not present the shear failure of bolts. The pioneer study in this regard was commenced by University of Wollongong via double shear test program in 2003, however additional studies require to be carried out to create an all-encompassing cable bolt database given certain ground conditions for the Australian coal mining industry.

The single shear testing methodology outlined in the BS7861-2:2009 and the key study conducted by Goris (1996), both failed to simulate the environments of the field failure. Aziz et al. (2015c) noted the results from the BS7861-2:2009 underestimated the shear performance with the expected hypothesis of single shear equalling half that of the double shear capacity was not reached between the direct shear test and the University of Wollongong double shear test. Shearing performance of cable bolts is required to optimise cables into the future to maximise their effectiveness to resist and redirect stresses, which is at the forefront of ensuring an integral excavation. The following chapter outlines a new methodology for the single shear test, which provides a more industry encompassing setting, addressing the deficiencies of the preceding studies.
Chapter 4: THE NEW SINGLE SHEAR TEST METHOD

4.1. Introduction

Megabolt designed a new single shear apparatus cable bolt test facility. Megabolt is an internationally acclaimed strata support products and systems provider. This new apparatus is designed to eliminate the shortage of outlined in British Standard (BS7861-2, 2007) for single shearing of cable bolts. The new design uses double embedment with a fully encapsulated cable in a cementitious material, which presents similar characteristics to rock mass, as per Goris (1996) and Craig and Aziz (2010).

4.2. Megabolt single shear testing apparatus

4.2.1. Components of the testing apparatus

The single shear instrument is horizontally aligned integrated system, consisting of the shearing rig and a 120t compression machine. The shearing cylinder is fabricated in two sections, each containing 1.8m of concrete anchor cylinder, providing a centrally located shearing plane. The shearing cylinder is enclosed in steel clamps to provide confinement during shearing. The shear displacement is applied through by four hydraulic rams, located at the bottom of the single shear rig, with the applied shear load measured by a pressure transducer and analogue gauge as shown in Figure 4-1, Figure 4-2 and Figure 4-3.

The hydraulic pressure rams are connected to one hose attachment which is also connected to a hydraulic pump. The rate of loading is applied through manual application, with an aim to apply a constant load in line with F 432-04 (ASTM, 2005) and BS 7861-2 (British Standard, 2009) of 1mm/min (0.018 mm/sec). Linear Variable Displacement Transducers (LVDT) was used to measure shear displacement at the shearing plane and any debonding at the cable ends. A data taker recorded the readings of the LVDTs at a constant time interval which were utilised for further data analysis.
Figure 4-1: Schematic of the Megabolt single shear apparatus (adopted from (Hawkins, 2016))

Figure 4-2: Single shear apparatus with a sample in position (adopted from (McKenzie and King, 2015))
4.3. Experimental study parameters

The following section presents the factors are effective on the performance of cable bolts subject to shearing. In the experimental method, all elements that effect shear are controlled and standardised across all samples.
Table 4-1: Types of cables tested ((Megabolt, 2016, Jennmar, 2016))

<table>
<thead>
<tr>
<th>Cable type</th>
<th>Longitudenal section</th>
<th>Cross Section</th>
<th>Bolt diameter (mm)</th>
<th>UTS strand (t)</th>
<th>Elongation at strand failure (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain MW10</td>
<td></td>
<td></td>
<td>31</td>
<td>70</td>
<td>5-6</td>
</tr>
<tr>
<td>Spiral MW9</td>
<td></td>
<td></td>
<td>31</td>
<td>62</td>
<td>5-6</td>
</tr>
<tr>
<td>Secura HGC</td>
<td></td>
<td></td>
<td>30</td>
<td>68</td>
<td>5-6</td>
</tr>
<tr>
<td>Indented SUMO-Hollow</td>
<td></td>
<td></td>
<td>28</td>
<td>63</td>
<td>5-7</td>
</tr>
<tr>
<td>Indented TG-Hollow</td>
<td></td>
<td></td>
<td>28</td>
<td>60</td>
<td>5-7</td>
</tr>
<tr>
<td>Garford Twin-strand</td>
<td></td>
<td></td>
<td>15.2</td>
<td>26.5</td>
<td>6.5</td>
</tr>
<tr>
<td>Superstrand Plain and Spiral</td>
<td></td>
<td></td>
<td>21.8</td>
<td>60</td>
<td>6-7</td>
</tr>
</tbody>
</table>
4.3.1. The strength of the grout bonding agent

The Minova Stratabinder HS cementitious grout was considered as bonding agent for this practical study. This kind of agent is a high strength thixotropic grout that is suitable for grouting method especially for the new modified Single and Double shearing apparatus to attach the cable to the concrete blocks. Stratabinder HS is low in viscosity, which can flow easier throughout the annulus area and it has minimal shrinkage characteristic (Orica, 2016). These properties enhance the bond strength between the cable and the concrete host material.

In order to reach to a W: C ratio of 0.35 to 0.4, Minova suggests that 20Kg of grout cementitious powder is required to be mixed with 6 to 8 litters of water. In this regard, all sixteen samples were encapsulated by the mixture of 20kg cement to 7.5kg of water to create W: C of 0.375 to achieve the nominated UCS strength of 60MPa. The samples cured for a time of 28 days as per Minova product specifications (Table 4-2) and as per previous UOW testing regarding UCS of grout in respect to curing time (Mirza et al., 2016) shown in Figure 4-4.

Table 4-2: Stratabinder HS product specifications (Minova 2016)

<table>
<thead>
<tr>
<th>Stratabinder HS product specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age (days)</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>7</td>
</tr>
<tr>
<td>28</td>
</tr>
</tbody>
</table>
1800KN Avery Compression testing machine was utilized to determine the Ultimate Compressive Strength of the grout samples. All samples collected from each grout mix and tested after a period of 28 days as shown in Figure 4-5. The average UCS obtained through the testing was 65.14MPa (Table 4-3).

Table 4-3: Stratabinder UCS test results (Orica, 2016)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Length 1 (mm)</th>
<th>Length 2 (mm)</th>
<th>Max Load (kN)</th>
<th>UCS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50.2</td>
<td>50.2</td>
<td>167</td>
<td>66.27</td>
</tr>
<tr>
<td>2</td>
<td>50.4</td>
<td>50.7</td>
<td>176</td>
<td>68.88</td>
</tr>
<tr>
<td>3</td>
<td>50.6</td>
<td>50.5</td>
<td>154</td>
<td>60.27</td>
</tr>
</tbody>
</table>

Average UCS 65.14
4.3.2. Diameter of rifled holes

42mm was set for the rifled hole diameters for all samples and as shown in Table 4-1, the diameter is different for each cable regarding the manufacturer design, in turn the difference between the borehole diameter and the cable diameter is filled with grout. Therefore, the annulus area for each cable type is calculated as follow:

- Plain MW10 and Spiral MW9 – 5.5mm
- Secura HGC – 6mm
- Indented TG and Indented SUMO – 7mm
- Garford-Twin Strand – 11.6mm
- Superstrand – 10.1mm

To ensure for full encapsulation the industry principal agrees with thickness of the grout to be considered between 4 and 10mm in order to allow the grout to run through easily

4.3.3. Pretension applied to the cable bolt

As previously explained, the British Standard (BSI 7861-1) does not allow cable bolts to be pretensioned, whilst the application of cable bolts in the field involves the pre-tensioning of cable, in other words, cable bolts are required to be placed some additional axial load. Table 4-4 outlines the experimental study plan for each cable geometry
including the presence of bulbs and the effect of pretension on the results of the cable to resist shear.

Table 4-4: Experimental plan of single shear testing of cable bolts

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Cable</th>
<th>Manufacturer</th>
<th>Design</th>
<th>Pre-tension load (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Plain MW10</td>
<td>Megabolt</td>
<td>No bulb</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>Plain MW10</td>
<td>Megabolt</td>
<td>6 bulbs</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>Plain MW10</td>
<td>Megabolt</td>
<td>6 bulbs</td>
<td>15</td>
</tr>
<tr>
<td>4</td>
<td>Spiral MW9</td>
<td>Megabolt</td>
<td>6 bulbs</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>Spiral MW9</td>
<td>Megabolt</td>
<td>6 bulbs</td>
<td>15</td>
</tr>
<tr>
<td>6</td>
<td>Spiral MW9</td>
<td>Megabolt</td>
<td>No bulb</td>
<td>15</td>
</tr>
<tr>
<td>7</td>
<td>Secura HGC</td>
<td>Minova</td>
<td>6 bulbs</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>Secura HGC</td>
<td>Minova</td>
<td>6 bulbs</td>
<td>15</td>
</tr>
<tr>
<td>9</td>
<td>Indented SUMO</td>
<td>Jenmar</td>
<td>6 bulbs</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>Indented SUMO</td>
<td>Jenmar</td>
<td>6 bulbs</td>
<td>15</td>
</tr>
<tr>
<td>11</td>
<td>Indented TG</td>
<td>Jenmar</td>
<td>No bulb</td>
<td>0</td>
</tr>
<tr>
<td>12</td>
<td>Indented TG</td>
<td>Jenmar</td>
<td>No bulb</td>
<td>15</td>
</tr>
<tr>
<td>13</td>
<td>Plain Sumo</td>
<td>Jenmar</td>
<td>6 bulbs</td>
<td>0</td>
</tr>
<tr>
<td>14</td>
<td>Plain Sumo</td>
<td>Jenmar</td>
<td>6 bulbs</td>
<td>15</td>
</tr>
<tr>
<td>15</td>
<td>Superstrand</td>
<td>Jenmar</td>
<td>No bulb</td>
<td>15</td>
</tr>
<tr>
<td>16</td>
<td>Garford</td>
<td>Jenmar</td>
<td>Bulbed</td>
<td>0</td>
</tr>
</tbody>
</table>

4.3.4. *Ultimate Compressive Strength of adjacent rock strata*

The 40MPa concrete blocks mixed with 10mm aggregates were utilised to meet the requirements to simulate the in situ behaviour of rock mass failure which presents both axial and tension forces during deformation. Moreover, concrete with minimal difference in ultimate compressive strength among all samples provide similarity.

Past testing, with particular mention to Clifford, Kent et al. (2001) and Thomas (2012), utilized sandstone core as the host material. This exhibits a more real imitation of the real world rock mass in the laboratory, however core drill size was limited and expense for the size required for each sample is not appropriate for this test.
4.3.5. **Embedment length**

The single shear test proposed by Mega bolt provides 900mm anchorage length into a cylinder concrete sample in each side of the shear joint. However, this embedment length clearly showed that the 900mm was inadequate to prevent plain cables from debonding. The Figure 4-6 illustrates that due to noticeable shear displacement that occurred at the shear joint, the lower embedment length would affect the bonding strength at the grout-cable interface.

![Debonding of cable during loading](image)

Figure 4-6: Debonding of cable during loading (adopted from (McKenzie and King, 2015))

As a matter of fact, it was suggested to double the length of the embedment (1800mm) in each side of the joint to evaluate the system if debonding will be occurred. It is worth pointing out that the partial debonding will be still occurring throughout the tests due to imposing a direct shear load, nevertheless the axial load known as tensile load and displacement are monitored through LVDT and digital gauge.

4.3.6. **Friction across the shear plane**

The principal aim of utilizing single shear test is that the shear failure of the cable to be evaluated. Therefore, it was attempted to eliminate the effect of the friction at the shear joint.

The principal aim of utilizing single shear test is that the shear failure of the cable to be evaluated. Therefore, it was attempted to eliminate the effect of the friction at the shear joint. In order to do this, a Teflon film was attached between the shear faces at the shear joint to prevent any contact during the test (Figure 4-7). Teflon has 0.04 friction coefficient which the created friction force of friction from such material is negligible.
4.4. **Test sample preparation**

4.4.1. *Casting and curing of cylindrical concrete blocks*

The moulds were adopted for casting the cylindrical concrete was Cardboard. It had 900mm height and 250mm wide. The borehole was created through inserting a 1000mm steel rod at the centre of the mould covered by 8mm diameter plastic conduit to simulate rifling. In addition, a plate at the bottom of each mould was accounted in order to keep the mould steady on the pallet, which has a 2cm thickness as it shown in Figure 4-8 and Figure 4-9.

The compressive strength of the concrete was aimed to be 40MPa included 10mm aggregate and it was made by Baine concreting services. Accordingly, slump tests were carried out to make sure the fresh concrete met the requirements, in conjunction with the consistency and flow-ability of the concrete.

The steel rod and plastic conduit was removed from each samples after 24 hours when the initial hardening of the concrete was reached. The cure time for the concrete samples were considered to be 28 days in order to attain 40MPa. In total, 64 concrete cylinders were cast over three batch of concrete pouring.
Schmidt hammer was utilized to confirm the nominated strength of the concrete (Figure 4-10). The overall Ultimate Compressive Strength of 45.5MPa (Table 4-5).
Table 4-5: Schmidt Hammer verification results

<table>
<thead>
<tr>
<th>Test Number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical readings</td>
<td>35</td>
<td>35</td>
<td>39</td>
<td>39</td>
<td>37</td>
<td>38</td>
<td>38</td>
<td>37</td>
<td>36</td>
<td>40</td>
</tr>
<tr>
<td>Lateral readings</td>
<td>49</td>
<td>42</td>
<td>44</td>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical average reading</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>37.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lateral average reading</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>43.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical UCS average (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>43.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lateral UCS average (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>48.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overall UCS (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>45.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.4.2. Adhesion of concrete anchor cylinder

As the embedment length was proposed to be 1800mm in this experimental study, two concrete cylinder, therefore, were needed to attach together to create the required length
on each side of the shear joint. The process of fixing and attaching the concrete cylinders were carried out in ‘Base Frame’ through using clamps as illustrated in Figure 4-11.

![Base Frame with clamped anchor cylinders](image)

Figure 4-11: Base frame with clamped anchor cylinders

In order to attach the shear joint interface, Epirez non-sag epoxy binder was utilized at a ratio of 3:1. As it can be seen from the Figure 4-12 the surface of each face was rubbed with epoxy mixture excluded the borehole to allow the grout running through easily.

![Application of epoxy on cylinder joint surface](image)

Figure 4-12: Application of epoxy on cylinder joint surface
Once each face of the sample were covered by epoxy, the load bearing plate push the concrete to be butted to the other face by applying compression load to the plate positioned at the corner of the Base Frame (Figure 4-13 and Figure 4-14). A confining pressure then applied through steel clamp to hold the samples together and after 30 minutes the compression load was removed to allow for epoxy to cure.

After preparing each leg of the test and curing was carried out, each pair were moved to another frame, named as the pretensioning and grouting frame to attach each pair together to create a 1800 mm sample. To do that, a clamp is required to be placed on the frame at the position of shear joint (Figure 4-16). The principal reason for this clamp was to maintain the integrity of the shear joint and also convenience of lifting the whole sample to transfer to the actual single shear apparatus to test.

Figure 4-13: Load bearing plate

Figure 4-14: Positioning of the anchor cylinder in the frame
It is noted that before the epoxy was rubbed to the shear face, a washer needed to be attached to the borehole to protect the Teflon and also not to allow the grout leaking out of the annulus. Table 4-6 shows the dimension of the Teflon and washer. The washer is attached through utilizing a Sika Bond Adhesive, a high strength contact spray. Afterward, Teflon is applied to the surface of the shear joint to prevent friction. The thickness of the Teflon was 2 mm and for each shear joint 2 Teflon was adopted to allow a 4 mm opening at the shear joint.

Table 4-6: Dimensions of teflon plate and washer seals

<table>
<thead>
<tr>
<th></th>
<th>Teflon plate</th>
<th>Washer seal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer radius (mm)</td>
<td>245</td>
<td>45</td>
</tr>
<tr>
<td>Inner radius (mm)</td>
<td>85</td>
<td>85</td>
</tr>
</tbody>
</table>

Figure 4-15: Application of washer seal and teflon plate at joint
4.4.3. *Cable bolt installation*

To monitor debonding strain gauges were attached on the cable bolts prior to installation of cables. The previous study conducted by Megabolt proved that the debonding occurs in the embedment length of 900mm and therefore no strain gauge was attached on the cable for the first 900mm of the cable. Instead all the strain gauges were aimed to be attached on the outer 900mm as it shown in Figure 4-17.

![Fixing the primary clamp to the shearing plane](image.png)

**Figure 4-16: Fixing the primary clamp to the shearing plane**

![Strain gauge locations](image.png)

**Figure 4-17: Strain gauge locations (adopted from (Hawkins, 2016))**

80
To attach the strain gauge, the aimed location required to be cleaned by alcohol based material and be sanded to be placed at the flat and even area (Figure 4-18). After that, the strain gauges were placed on the cable and attached to the cable through Araldite epoxy resin. The cure time for adhesive was 2 minutes.

![Application of strain gauge](image)

Figure 4-18: Application of strain gauge

Each strain gauge was connected to the data logger through a low resistance wire. In this study, in order to ensure that such fragile wire would not damage throughout the test, Silastic was utilized to cover the wires, however it was significant to minimize the amount of silastic as it might affect the bonding strength of the cable-grout interface.

Once the strain gauges were applied and fixed, the cable carefully was installed into the borehole. It is noted that the pretensioning applied to the side of the cable that was not facilitated with strain gauges to protect the wires to avoid damaging during pretensioning process. A spacer and nut is required for the side of the sample needs grouting and 62.5mm barrel and wedge is required for the other leg of the sample where axial load was imposed (Figure 4-20). Pretension load was monitored through a dial gauge before the pretension machine being released. Figure 4-19 shows the pretension equipment.
4.4.4. *Grouting the cable*

The grouting samples were carried out through bottom up methodology. In fact, the frame of pretensioning and grouting was angled to 65° before the grouting process was commenced (Figure 4-21). In this methodology grout is pumped from one leg to another leg to fill the annulus. The advantage of such method is that the provided angle reinforces a weight force induced by the gravity to prevent any air bubbles forming.
The Minova Stratabinder HS cementitious grout was utilized as bonding agent for this practical study as described in Section 4.3.1. in a paddle mixer. A mixer that used for this practical study to provide an appropriate grout was a Paddle mixer with a piston pump since it met the industry requirements. The curing time for the sample was considered a minimum of 28 days at the aforementioned angle to achieve a 72MPa of ultimate compressive strength for testing.

4.5. Testing procedure

After curing time is reached all samples were disassembled from the frame and transported to the actual shearing apparatus. Once the 3600mm was assembled properly through the shearing apparatus, steel clamps were installed on the sample to provide confinement pressure on the sample to simulate the in situ environment.

As it illustrated in Figure 4-22 all the previous accessories such as nut, spacer, barrel and wedge and grouting trumpets were removed from the sample to allow the LVDT being set up at the end of each leg. The installed LVDT provided a numerical value for displacement.
The data logger was utilized to monitor and record all the data from the pressure transducer, LVDT as well as strain gauges (Figure 4-22). A hydraulic power pack was connected the hydraulic rams and to commence the testing, the loading process was carried out manually and attempted to keep the rate of the loading to a constant rate of 1mm/min. The load was applied until complete failure of the cable was reached or full debonding is occurred.

4.6. Chapter summary

In this chapter, the new single shear test methodology was presented to closely simulate the behaviour of cable bolts in shear. In this method, pretensioning capability of the new single shear apparatus was introduced. This apparatus allows the debonding to be monitored and the numeric values are recorded to create a more credible and reliable results. However, the difficulty of this methodology revealed throughout the chapter shows it to be requiring specific expertise, tedious program and an extended time to collect an appropriate sample size.

Figure 4-22: LVDTs connected to the extremity of the cable
Chapter 5: DOUBLE SHEAR TEST METHOD

5.1. Double shear testing apparatus

A very common test among researchers in order to verify and examine the shear strength of cable bolts is double shear test. In this chapter using new MKIII (Figure 5-1) is introduced and the preparation and procedure of the test is outlined.

![Diagram of double shear testing apparatus]

5.1.1. Component of the testing apparatus

The double shear apparatus is horizontally aligned integrated system, consisting of three blocks of concrete with predetermined dimensions. The outer blocks have the same dimension as 300x300mm² while the central block is rectangular with 450mm in length. The shearing blocks are enclosed in steel plates in order to provide confinement throughout shearing. Eight steel bars are installed in order to prevent the outer concrete blocks from overbalancing and falling down during shearing. Moreover, the side steel plates are enclosed to allow horizontal braces to be installed to control the gap in shear face and also impede the normal load on concrete during shearing. Inside and outside palates are installed for hold the load cell (60 tonne) in order to remain steady after pre-tensioning and also in case of high pressure load from pre-tensioning the inside and outside plates acting as reinforcement to prevent the side plates from bending. All three
concrete blocks seat on a base platform which fits with the bottom ram of the 500 tonne compressions Test machine. Barrel and wedge is applied at either ends to constraint cable. The outer blocks are seated on three thick plates (110mm) in order to make space for vertical displacement of central steel frame shear box, and also the middle blocks are seated on a removable plate in order to make the system balanced horizontally. After the double shear apparatus for test being prepared, the apparatus is placed on the bottom ram of the 500 tonne compression test machine. The machine is set on applying shear displacement load on the rate of 1mm/min, as shown in Figure 5-2.

![Figure 5-2: Tensioned and grouted DST assembly ready for cap closing](image-url)
5.2. **Experimental study parameter**

In this section the factors affect the shear strength performance of cable bolt is controlled and studied.

5.2.1. *Type of cable bolt*

The verity of cable bolts are tested in this experimental work of research as shown in Table 5-1 to examine the shear strength performance of such cables utilized in Australian mining.

Table 5-1: Type of cable bolt as tested in this study (Megabolt, 2016)

<table>
<thead>
<tr>
<th>Cable type</th>
<th>Longitudinal section</th>
<th>Cross section</th>
<th>Bolt diameter (mm)</th>
<th>UTS strand (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain MW10</td>
<td><img src="image1.png" alt="Image" /></td>
<td><img src="image2.png" alt="Image" /></td>
<td>31</td>
<td>70</td>
</tr>
<tr>
<td>Spiral MW9</td>
<td><img src="image3.png" alt="Image" /></td>
<td><img src="image4.png" alt="Image" /></td>
<td>31</td>
<td>62</td>
</tr>
<tr>
<td>SUMO-Plain</td>
<td><img src="image5.png" alt="Image" /></td>
<td><img src="image6.png" alt="Image" /></td>
<td>28</td>
<td>65</td>
</tr>
<tr>
<td>Sumo-Indented</td>
<td><img src="image7.png" alt="Image" /></td>
<td><img src="image8.png" alt="Image" /></td>
<td>28</td>
<td>63</td>
</tr>
</tbody>
</table>
5.2.2. Strength of the grout bonding agent

In this experiment Minova Stratabinder HS cementitious grout was chosen for all the tests. Further information is supplied in Section 4.3.1.

5.2.3. Diameter of rifled holes

The borehole diameter as known rifled hole diameter was set to 42mm through all tests as shown in Table 5-1. According to manufacturer the cable bolts diameters are varied as shown in Table 5-2 above, which leads the annulus area being different for each cable. The annulus area for each cable types:

- Plain and Spiral MW10-MW9 – 5.5mm
- Plain and Indented SUMO – 6mm

Table 5-2: Single shear testing cables characteristics

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Cable</th>
<th>Manufacturer</th>
<th>Pre-tension load (t)</th>
<th>Drill bit (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Plain MW10</td>
<td>Megabolt</td>
<td>6</td>
<td>42</td>
</tr>
<tr>
<td>2</td>
<td>Plain MW10</td>
<td>Megabolt</td>
<td>15</td>
<td>42</td>
</tr>
<tr>
<td>3</td>
<td>Spiral MW9</td>
<td>Megabolt</td>
<td>0</td>
<td>42</td>
</tr>
<tr>
<td>4</td>
<td>Spiral MW9</td>
<td>Megabolt</td>
<td>7.5</td>
<td>42</td>
</tr>
<tr>
<td>5</td>
<td>Spiral MW9</td>
<td>Megabolt</td>
<td>15</td>
<td>42</td>
</tr>
<tr>
<td>6</td>
<td>Plain- SUMO</td>
<td>Jenmar</td>
<td>0</td>
<td>42</td>
</tr>
<tr>
<td>7</td>
<td>Plain- SUMO</td>
<td>Jenmar</td>
<td>15</td>
<td>42</td>
</tr>
<tr>
<td>8</td>
<td>Spiral-SUMO</td>
<td>Jenmar</td>
<td>0</td>
<td>42</td>
</tr>
<tr>
<td>9</td>
<td>Spiral- SUMO</td>
<td>Jenmar</td>
<td>15</td>
<td>42</td>
</tr>
</tbody>
</table>

As a matter of fact, the thickness of the grout in the annulus is 0.5mm different between MW9-10 and SUMO.

5.2.4. Pretension applied to the cable bolt

As previously explained, the British Standard (BSI 7861-1) only allows for the passive testing of cable bolts. Industry application of cable bolts involves the pre-tensioning of cable, which is adding an axial load to the cable. Table 5-2 outlines the experimental
study plan for each cable geometry and the effect of pretension on the results of the cable to resist shear.

5.2.5. UCS adjacent rock mass

A 40MPa concrete with 10 mm aggregates was conducted for this experimental study. In situ rock mass characteristic is replicated by concrete cementitious based material as clearly represents the axial load forces during deformation. Furthermore, the homogeneity of concrete materials provides a minimal difference of ultimate tensile strength.

Past testing, with particular mention to Clifford, et al. (2001) and Thomas (2012), utilised sandstone core as the host material. This is a truer representation of real world application however the restrictiveness of core drill size and expense for the size required for each sample is not appropriate for this test.

5.3. Test sample preparation

Three concrete blocks with cross-sectional area of 300x300mm$^2$ are cast for each double shear test. The difference between three concrete blocks is in the length, the two outer blocks are 300mm long while the central block is 450mm.

Figure 5-3 casting the concrete is carried out into three separate steel frames of double shear apparatus with thickness of 20mm. While the concrete is mixing, the mould is being prepared to pour the concrete in the mould. All frames are seated on 10mm wooden platform and also clamp is utilized for splint the mould. Moreover, four wooden barriers are used to stabilize as well as separating the blocks from each other, as it is shown in Figure 5-4.
Prior to casting, a plastic conduit is also positioned through the moulds for cable bolt installation with 3mm diameter electrical wire wrapped around the conduit to create a rifled borehole for cable anchorage. Once assembling the mould is finished, greasing is required in order to allow concrete to be removed easier after the concrete set as grease prevents sticking concrete to the mould. The grease type utilised is petroleum jelly.

Casting the concrete starts with mixing the half of the total amount of concrete in the mixer. Approximately 3 minutes is specified for mixing the cement and sand and then water is added. Afterward allows the mixer to mix the materials for 5-6 minutes. Once mixed, concrete is poured into each section of the mould and in the meanwhile an electric vibration is utilized in order to eliminate the air bubble to make the concrete uniform and increase its strength. The second part is mixed immediately after finishing the first part and the same steps are repeated. After the mould was filled, immediately
three plastic tubes are inserted into the surface of the concrete blocks to make inlet holes for grouting. Figure 5-5 clearly representing the above mentioned procedure.

From the same batch, three cylindrical concrete samples (Figure 5-6) consisting of a height and diameter of 200mm and 100mm are utilized in order for determining the Unconfined Compressive Strength (UCS) of the concrete. The cylindrical moulds were filled in two steps as it needs to be stucked 25 times in each half to make the concrete uniform.
The concrete was left in mould for 24 hours to get harden and then removed and stored in laboratory condition to cure for approximately 30 days to reach the nominal strength. Three cylindrical samples were also removed and stored in the water pond at the laboratory to cure.

5.3.1. *Unconfined Compressive Strength (UCS) test*

The strength of the concrete was confirmed through the usage of compression testing machine (Figure 5-7) in the laboratory of the University of Wollongong after minimum 28 days curing. After breaking the cylindrical concrete samples the results returned an overall UCS test of 45MPa.
5.4. **Cable bolt installation**

After minimum of 28 days curing time for concrete blocks passed, the concrete blocks would be ready to be examined (Figure 5-8). Accordingly the concrete blocks were placed back into the steel frames and seated on the carrier platform and related plates. Then a cable with desired length was placed through the concrete hole. It is significant to make a gap between the blocks in order to avoid friction or any contact between shear planes. Since there are gaps between blocks, a plastic joint ring was laid on the cable to lead grout running through the gaps. Then side steel plane was attached at either proximity to allow lateral braces being enclosed. Once the concrete blocks and lateral braces were fixed the outer plates as well as 60 tonne load cells installed. At the end in accordance with the amount of pre-tensioning load (Table 5-2), the cable was pre-tensioned through a hydraulic tensioner and barrel and wedges at either end of the cable, during the pre-tensioning the load cells were connected to the data-taker to allow monitoring the amount of pre-tensioning. As soon as pre-tension load is applied and reached the required value, the grout was mixed and pours into the concrete through inlet hole. In order to prevent escaping the grout from the either end of the borehole, masking tape as well as silicone gel was utilized. Then the sample was left for at least 7 days to cure.
5.5. **Testing procedure**

Once the mandatory time for curing the grout was reached the sample would be ready to be lifted precisely into shearing rig and placed on the 600x 600mm$^2$ loading platen of the 500T compression testing machine as shown in Figure 5-9. Before positioning the sample on the shearing rig, it is required to place the steel plates on the samples and tighten them up in order for providing a confinement pressure during shearing. It is noted that all three steel frame sections of the sample were seated on about 110mm high steel and timber plates when assembling the concrete blocks. However, prior to testing and loading the apparatus, the timber plates beneath the central block was removed to allow the central block to move vertically down during the shearing process. The applied displacement load on central block was set to the rate of 1mm/min and was controlled during the test process. Moreover, the vertical movement of the central block was limited into 100mm for each test. The compression machine embedded recording system was utilized to record the shear load and shear displacement and the axial load.
was recorded by data-taker hooked up to the load cells. The shearing load, displacement and axial load was monitored during loading throughout the test and it was possible to be visually seen on the computer screen.

![Figure 5-9: View of double shear box testing](image)

5.6. **Chapter summary**

In this chapter the new double shear testing methodology with third generation MKIII outlined to examine the shear strength of the cable bolt with no friction effect at the joints. This method provides the capability of applying pre-tension load to the cable and monitoring the shear load, displacement and axial load during the test.
Chapter 6: RESULTS AND DISCUSSION

6.1. Test results

In this chapter, the data achieved from all the 16 single shear tests as well as 9 double shear tests which are summarized in Table 6-1 and Table 6-2. In these tables, peak shear load for each cable bolts with related displacement are presented. In addition to that Table 6-1 demonstrates the displacement pertinent to debonding of the cables. Further exploration of findings will be covered on following sections in terms of cables configuration and geometry and their impact on load performance in relation to peak shear load, shear displacement and occurrence of any debonding of the sample. The wire failures will be analysed in this chapter to determine if the geometry of a wire strand induced a particular failure mode.

All results from single shear test are compared to results of new double shear test programme conducted in University of Wollongong with new frictionless apparatus MKIII to validate and reliability between all the sample sets.

Table 6-1: Brief results of single shear test

<table>
<thead>
<tr>
<th>Test</th>
<th>Cable</th>
<th>Design</th>
<th>Pretension (t)</th>
<th>Peak shear load (t)</th>
<th>Shear displacement at peak shear (mm)</th>
<th>Cable debonding</th>
<th>Peak shear load/ UTS (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Plain MW10</td>
<td>No bulbs</td>
<td>15</td>
<td>68.34</td>
<td>68.24</td>
<td>Yes</td>
<td>97.6</td>
</tr>
<tr>
<td>2</td>
<td>Plain MW10</td>
<td>6 bulbs</td>
<td>0</td>
<td>63.84</td>
<td>52.57</td>
<td>Yes</td>
<td>91.1</td>
</tr>
<tr>
<td>3</td>
<td>Plain MW10</td>
<td>6 bulbs</td>
<td>15</td>
<td>60.39</td>
<td>56</td>
<td>Yes</td>
<td>86.3</td>
</tr>
<tr>
<td>4</td>
<td>Spiral MW9</td>
<td>6 bulbs</td>
<td>0</td>
<td>47.73</td>
<td>43.5</td>
<td>No</td>
<td>76.9</td>
</tr>
<tr>
<td>5</td>
<td>Spiral MW9</td>
<td>6 bulbs</td>
<td>15</td>
<td>52.6</td>
<td>41.4</td>
<td>NO</td>
<td>84.8</td>
</tr>
<tr>
<td>6</td>
<td>Spiral</td>
<td>No bulbs</td>
<td>15</td>
<td>49.7</td>
<td>41.73</td>
<td>No</td>
<td>67.3</td>
</tr>
<tr>
<td></td>
<td>MW9</td>
<td>Description</td>
<td>Count</td>
<td>Time</td>
<td>Efficiency</td>
<td>Is Bulbs</td>
<td>Code</td>
</tr>
<tr>
<td>---</td>
<td>---------</td>
<td>------------------------------</td>
<td>-------</td>
<td>------</td>
<td>------------</td>
<td>----------</td>
<td>------</td>
</tr>
<tr>
<td>7</td>
<td>Secura</td>
<td>Plain and Spiral strands</td>
<td>0</td>
<td>64.69</td>
<td>53.86</td>
<td>No</td>
<td>95.2</td>
</tr>
<tr>
<td>8</td>
<td>Secura</td>
<td>Plain and Spiral strands</td>
<td>15</td>
<td>55.9</td>
<td>47.8</td>
<td>No</td>
<td>82.2</td>
</tr>
<tr>
<td>9</td>
<td>Indented SUMO</td>
<td>Birdcage</td>
<td>0</td>
<td>46.42</td>
<td>48.31</td>
<td>No</td>
<td>73.7</td>
</tr>
<tr>
<td>10</td>
<td>Indented SUMO</td>
<td>Birdcage</td>
<td>15</td>
<td>37.35</td>
<td>32.83</td>
<td>No</td>
<td>59.4</td>
</tr>
<tr>
<td>11</td>
<td>Indented TG</td>
<td>No bulbs</td>
<td>0</td>
<td>31.09</td>
<td>35.4</td>
<td>No</td>
<td>51.81</td>
</tr>
<tr>
<td>12</td>
<td>Indented TG</td>
<td>No bulbs</td>
<td>15</td>
<td>36.31</td>
<td>34.4</td>
<td>No</td>
<td>60.51</td>
</tr>
<tr>
<td>13</td>
<td>Plain SUMO</td>
<td>6 bulbs</td>
<td>0</td>
<td>54.7</td>
<td>71.8</td>
<td>Yes</td>
<td>86.8</td>
</tr>
<tr>
<td>14</td>
<td>Plain SUMO</td>
<td>6 bulbs</td>
<td>15</td>
<td>67.1</td>
<td>78.2</td>
<td>Yes</td>
<td>106.3</td>
</tr>
<tr>
<td>15</td>
<td>Super Strand</td>
<td>Plain 19 wire cables</td>
<td>15</td>
<td>51.4</td>
<td>90.2</td>
<td>Yes</td>
<td>85.7</td>
</tr>
<tr>
<td>16</td>
<td>Garfordd</td>
<td>Twin-strand</td>
<td>0</td>
<td>43.7</td>
<td>46.8</td>
<td>Yes</td>
<td>80.9</td>
</tr>
</tbody>
</table>
Table 6-2: Brief results of double shear test

<table>
<thead>
<tr>
<th>Test</th>
<th>Cable</th>
<th>Design</th>
<th>Pretension (t)</th>
<th>Peak shear load (t)</th>
<th>Shear displacement at peak shear (mm)</th>
<th>Peak shear load/UTS (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SUMO</td>
<td>Plain</td>
<td>0</td>
<td>88.59</td>
<td>100</td>
<td>68.1</td>
</tr>
<tr>
<td>2</td>
<td>SUMO</td>
<td>Plain</td>
<td>15</td>
<td>85.2</td>
<td>88.2</td>
<td>65.5</td>
</tr>
<tr>
<td>3</td>
<td>SUMO</td>
<td>Spiral</td>
<td>0</td>
<td>81.49</td>
<td>93.43</td>
<td>64.7</td>
</tr>
<tr>
<td>4</td>
<td>SUMO</td>
<td>Spiral</td>
<td>15</td>
<td>76.7</td>
<td>85.74</td>
<td>60.9</td>
</tr>
<tr>
<td>5</td>
<td>MW10</td>
<td>Plain</td>
<td>0</td>
<td>87.88 (83.79)</td>
<td>105(100)</td>
<td>62.7</td>
</tr>
<tr>
<td>6</td>
<td>MW10</td>
<td>Plain</td>
<td>15</td>
<td>92.31</td>
<td>88.5</td>
<td>65.9</td>
</tr>
<tr>
<td>7</td>
<td>MW9</td>
<td>Spiral</td>
<td>0</td>
<td>93.89 (84.76)</td>
<td>114.5(100)</td>
<td>75.7</td>
</tr>
<tr>
<td>8</td>
<td>MW9</td>
<td>Spiral</td>
<td>7.5</td>
<td>90.7</td>
<td>89.7</td>
<td>73.1</td>
</tr>
<tr>
<td>9</td>
<td>MW9</td>
<td>Spiral</td>
<td>15</td>
<td>83.7</td>
<td>88.47</td>
<td>67.5</td>
</tr>
</tbody>
</table>

6.2. Load transfer performance of Plain MW10 cable bolt

Test 1, 2 and 3 illustrate the load transfer performance of the plain MW10 with different bulb structure and pretension values manufactured by Megabolt. This type of cable consists of a king wire covered by 10 plain wires. The UTS of 70t is allocated for this kind of cable which is the highest among the cables tested and also MW10 has one more wire compared to other cables tested, so the other cables have 9 wires. The MW10 sample results are depicted on a load-displacement graph in Figure 6-1.

Test 1 in Table 6-1 indicates that plain MW10 with no bulb structures and with an applied 15t pretension recorded a maximum peak shear load of 68.34t with a corresponding shear displacement of 68.24mm. Debonding was completely occurred at the cable-grout interface which was sourced from observing the dial gauges readings as well as centrally located LVDTT readings and strain gauges reading (Figure 6-1).

Test 2 presents the impact of bulbled structures with no pretension applied on the Plain MW10 cable. The peak shear load for this cable is 63.84t with a 62.57mm shear...
displacement and 40mm of the displacement sourced directly from the debonding. The effect of the bulbed structure was also explored in test 3 whilst with a 15t applied pretension. The cable returned a peak shear load of 60.39t, which was lower than it was expected, with an associated shear displacement of 56mm.

As it can be seen from the Table 6-1, the application of pre-tension was reduced the peak shear load by 5.4%, while the shear displacement was reduced from 62.57mm to 56mm which means an 11.7% decreased. So this can be a start for the idea that there is an inverse relationship between applied pretension and peak shear load and shear displacement. In this chapter the validation of such a relationship will be discussed.

Figure 6-1: Plain MW10 test results
Figure 6-5 shows the debonding for all samples because of shear displacement in peak shear load. So as it is clear, all the plain MW10 cables experienced the debonding in compare to other samples which returned the value of zero for debonding. So this can prove that the plain configuration reduces the bond strength at the cable-grout interface and this is because of surface roughness deficiencies.

Test 2 and 3 as are shown on the load-displacement graph in Figure 6-1, to have a similar immediate stiffness. As discussed further in this chapter, stiffness of the system can be increased by imposing pretension which leads to a lower elastic modulus. It can be seen from the data on the load-displacement graph that there is an undistinguishable change for the initial 5mm of displacement is sourced from the debonding present in the cables.

Figure 6-2 shows a schematic of shear interface of each sample after complete failure to be able to understand deeply the performance of each wire against shear and tensile load. The presence of cup and cone format at the end of each wire after failure, can reinforce the fact that the tensile load as an axial load was developed through each wire and ended up reaching their ultimate tensile strength, whereas shear failure was recognized through a near vertical failure surface, and a 45° surface can prove that a failure occurred with combination of shear and tensile. Whether the surface geometries of wires induced a certain failure mode is an attempt to investigate through all shear interfaces, all test photographs and schematics generated are shown in appendix A and B.
After analysing the plain MW10 with bulbed structures, test 2 and 3, it was seen that the majority of the wires failed in pure shear, whilst the majority of the wires with no bulb strand in Test 1, failed in tension. It is also worth to note that the excessive debonding occurred in tests was affected the failure modes.

Figure 6-2: Analysis of the failure mode of wires in Plain MW10

As it can be seen, the length of each wire at failure is different and this can be indicative of excessive debonding carried out through the application of the shear force. In fact, the noticeable displacement of the cable put an extra tensile load on the cable and therefore has led to increasing a considerable amount of tension on the cable and led to failure of approximately 80% of the strand in pure tension. It can be therefore mentioned that the present failure modes can be influenced by the loading applied on the system with a plain wire strand.

Figure 6-3: Test 1 - MW10 NB 15 tonne pretension - strand failure on the left anchor cylinder
Figure 6-4 and Figure 6-5 represent the amount of debonding that all MW10 cables underwent and it emphasizes the shortage of embedment length of anchor cylinder for the plain strand cables. As a matter of fact, the application of such cable in industry scale should be considered for lengths greater than 3.6m. In this sense, Megabolt provides cables in lengths from 4 to 13m, which therefore more investigation in the reliability of such cables is required to be conducted.

Figure 6-4: Test 1 strain gauge and LVDT measurements (Plain MW10 with no bulbs and 15 tonne pretension load)
Shear displacement corresponding to debonding (mm)

Test Number

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Shear Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>43.5</td>
</tr>
<tr>
<td>3</td>
<td>30</td>
</tr>
</tbody>
</table>

Figure 6-5: Shear displacement corresponding to debonding at the cable-grout interface for MW10 samples

6.3. Load transfer performance of Spiral MW9 cable bolt

Test 4, 5 and 6 illustrates the load transfer performance of the spiral MW9 manufactured by Megabolt. The UTS of 62t is allocated for the spiral strand with an elongation at strand failure of 5-6%. The tests conducted on spiral cables with and without bulbed structures as well as 0 and 15t pretension values to explore the effect of such structures on transferring load to make a direct comparison to the effect of spiral geometry to the plain configuration of the MW10. Figure 6-6 depicts the all load and displacement data for the spiral MW9. It is important to note that no MW9 experience debonding throughout the test as a result of shear displacement at peak shear load.

Test 4, spiral MW9 with bulbed structure and no pretension appeared to have a peak shear load of 47.73t with an associated displacement of 43.5mm. Test 5, the Spiral MW9 with bulbed structure and 15t pre-tension returned a peak shear load of 52.6t with 41.4mm shear displacement. Test 5 specified the highest value for peak shear load and the lowest displacement value among the all MW9 tests. These two tests, 4 and 5 do not confirm the outcomes of the plain MW10, which showed the pre-tensioned sample having a lower peak shear load. The theory is that applying pre-tension load on the cable leads to an increase at the stiffness of the system and this is because of reducing the flexibility of the cable. Furthermore Figure 6-6 indicates near identical elastic region
displacement for both the tests where Test 4 should gain greater displacement in the elastic zone. The stiffness in Test 4 is considered to be sourced from the crushing of the brittle host material during initial loading affecting the results.

![Spiral MW9 test results](image)

Test 6, finally, performed the load transfer of no bulbed structure with 15t pretension on spiral MW9 which obtained a peak shear load of 49.7t and a displacement of 41.73mm. With comparing the results for test 5 and 6 can be concluded that the role of bulbed structure is not significant on spiral MW9 sample as to have a similar peak shear load and only 0.7% improvement in shear displacement.

Figure 6-7 shows that the failure modes of spirally profiled wire strands in the MW9 are in tension, with the exception of test 6 with shear accounting for majority of the wire failures.
Finally, the loading performance of MW9 cables was not significantly influenced by additional bulbs. Naturally, the spiral cable creates a greater bonding strength at the cable-grout interface due to its roughness surface. Therefore, in case of spiral cables, the additional bulbs did not affect the load performance in comparison with plain MW10 cables through comparing the displacement. Moreover, in average, the plain MW10 achieved 90% of its UTS while spiral MW9 achieved 75% of its UTS. However, it is worth pointing out that the results for MW10 has been affected by debonding.

6.4. Comparison of the plain and spiral cable configurations on the loading performance of cable

By comparing results of MW10 and MW9, as shown in Figure 6-8, the effect of the spiral configuration can be analysed. Since the cable profiles are different in bulbed structure as well as pretension value, it is worth to compare those cables in which have the equal situation, except the geometry.

- Test 1 and Test 6
- Test 2 and Test 4
- Test 3 and Test 5

Test 1 and Test 6 performed with no bulbed structure and 15t of applied pre-tension load to the MW10 and MW9 samples respectively. MW10 with the former variables gained a peak shear load 37.5% higher than the MW9 counterpart. The MW9 was beneficial for lower shear displacement, reducing the displacement at peak by 63.5%, from 68.24mm in the MW10 to 41.73mm in the MW9 sample.
Test 2 and Test 4 performed with bulbs on the cable geometry with zero pre-tension applied. Likewise, MW9 recorded a lower shear displacement at peak load from 19.07mm, a reduction of 43.8%. Moreover, the peak shear load for the MW9 was greatly reduced from 63.84t to 47.73t.

Similarly, by comparison the Test 3 and Test 5 can be seen that the peak shear load on the plain wired cable is 14.8% higher than that in the spiral configuration, whilst the spiral configurations had a lower displacement. Plain MW10 had a 26.1% higher shear displacement at maximum shear load than the spiral MW9. The peak shear load for the Plain MW10 samples are higher than the spiral MW9 samples and this is because of having a higher Ultimate Tensile Strength value which can affect the reliability of MW9 cables adversely. Aziz, et al. (2014) claimed that when the cable is spirally configured a 10-12% of the material mass is lost, which reduces the ultimate tensile strength.

![Comparison of the Plain and Spiral wire configuration](image)

Figure 6-8: Comparison of the Plain and Spiral wire configuration

To conclude, according to Figure 6-9 among the all six tests Plain MW10 proved that it has a potential to resist higher shear load before failure, whilst the Spiral MW9 samples offered a lower shear displacement. Consequently, depending on the conditions of the strata if the minimal movement is concerned so according to data spiral MW9 would be more appropriate option and if higher peak shear load is required, plain MW10 would therefore be more efficient.
It is important to mention that due to spiral geometry of MW9 cables, none of them underwent debonding as spiral features of such cables provide a greater interlocking system between grout and cable interface.

6.5. The effect of bulbed configurations on the loading performance of the cable

The loading performance of the plain MW10 cable with and without utilizing bulbed structure can be compared through test 1 and 3 with only 15t pretension applied. Using bulbed structure on the cable MW10 decreased the maximum shear load by 11.6% and the shear displacement from 68.24 mm to 56 mm as shown in Figure 6-8.

The effect of bulbed structure on the loading performance on MW9 can be analysed through comparing tests 5 and 6. The visual depiction in Figure 6-8 indicate the application of bulbs on the spiral profile cable has negligible effect on the performance of the cable to resist shear force and the associated displacement at peak shear load.

To conclude, bulb structures was shown to be more effective on plain wires rather than spiral wires as the latter cables presented almost no change on the loading performance. Furthermore, the application of bulbed structure on Plain MW10 affected the peak shear load as well as shear displacement adversely. This is due to a rigid structure of that the bulb provides.
6.6. Load transfer performance of Secura HGC cable bolt

Test 7 and 8 explored the load transfer ability of Secura Hollow Groutable Cable (HGC) with and without the application of pretension (Figure 6-10). This type of cable includes 4 spiral wires along with 5 plain wires which in total nine wires wrapping around the king wire. The Secura HGC has the UTS of 68t which is the second highest UTS strength among the all samples.

Test 7, Secura HGC without the application of pretension to the cable appeared to have a peak shear load of 64.69t with associated displacement of 51.8mm. On the other hand, Test 8 performed with 15t axial load applied to the cable which returned a peak shear load of 55.9t and a shear displacement of 45.9mm. These two tests agree with the inverse relationship indicated in Test 1, 2 and 3 as the pretension load increases, the maximum shear load decreases.

Figure 6-10: Secura HGC test results
As it can be seen from the Figure 6-10, Test 8 reaches the elasto-plastic region at a more rapid rate in comparison with Test 7. This is because of the application of pretension on the cable. As it was mentioned before, applied pre-tension load affects the stiffness of the cable and it leads to a reduction on peak shear load due to reduced flexibility of the cable. Figure 6-10 has divided into 5 regions to make the analysis easier to follow. Region A is known as the elastic region. This area of the graph is associated with the elastic behaviour of the cable and it means that if the applied load removed the sample will return to its initial state. Once a sample exceeds its deformation, it automatically enters to a transitional section which is known as elasto-plastic (Region B). This region is placed between elastic and plastic state. In this section a sample will sustain some deformation after the load being removed. This area can be verified by the linear section after the elastic region where the rate of the shear stiffness of the system drops considerably. Plastic region is the ultimate state of each material which refers to the yield point of materials where the rate of the stress starts being low and the permanent deformation commence occurring. Data suggests that both samples entered to plastic region at approximately 15mm of displacement. It is important to note that from the yield point and beyond all deformation is non-reversible.

Figure 6-11: Analysis of failure of strands of Test 7 and 8 (Secura HGC)
In addition, Point D and E in Figure 6-10 illustrates the sample failure modes, meaning that strand wires start snapping which ended up leading to complete failure. Test 8 claims that when the first wire strand snapped the structural reliability of the cable starts to decline. According to Figure 6-12 and Figure 6-11, it can be understood that the bottom 5 wire strands failed at Point D due to smooth surface failure, moreover, it is probable that the remaining 4 strands on the top of the cable failed at Point E as there is jagged appearance on the two wires on the top of the cable.

![Figure 6-12: Test 7 cable failure on right anchor cylinder](image)

To conclude, the performance of Secura HGC in shear load is similar with plain MW10 cable. It is understandable from comparison of Secura and MW10 to MW9 that the existence of plain wires in the strand affects the UTS of the strand enormously which leads to an increasing the peak shear load of the cable. It is significant to point out that the Secura HGC did not undergo any debonding and this is because of irregularity of the cable strand surfaces which is a combination of plain and spiral. The study suggests that the combination of the two profiles, plain and spiral, is advantageous in terms of excluding debonding without adversely affect the strength of the cable.

6.7. Load transfer performance of Indented SUMO cable bolt

Test 9 and Test 10 show the loading performance of the indented SUMO cable, manufactured by Jennmar with and without application of pre-tension on the cable. The UTS for this type of cable, which is the indented 28 mm SUMO-Hollow strand with 35mm diameter birdcage, is 63t. The shear load-displacement graph illustrates the performance of such cables with zero and 15t applied pretension (Figure 6-13).
Test 9, Indented SUMO with zero pretension, gained the peak shear load of 40.43t with a corresponding displacement of 40.91mm. Test 10, Indented Sumo with applied 15t pretension on the sample, appeared to have a reduction on peak shear load which turned to 37.36t and significantly lower displacement of 30.9mm.

The inverse relationship between the applied pretension load and peak shear load can still be displayed by SUMO cable. The addition of 15t pretension on SUMO cable led to the displacement declined by 32.3% in comparison with SUMO without applied pretension load. A change of 8.2% on the peak shear load of the sample with 15t pretension appeared.

Figure 6-14 depicted that majority of wires for the zero pre-tension sample failed in tensile and combination of tensile and shear. A greater tensile force can be applied to the sample due to its additional movement sourced from its flexibility. On the other hand, the majority of wire failed in shear for the pre-tensioned sample. The displacement of the pre-tensioned sample was decreased by the stiffness in the sample and this has resulted in a reduction in the tension force due to the direct shear load. This further supports that the wire geometries have little effect on the failure mode, rather the type and extent of loading imposed on the system.
Conclusively, as data suggests that by placing pretension load on SUMO cables the reliability and integrity of the excavation increases. By comparing Plain Sumo cables to Secura cable bolts, imposing pretension load can be mentioned that has a detrimental effect to the reliability of the Secura cable bolt as the peak shear load decreased. However, shear displacement for Secura cable reduced whilst the corresponding shear displacement for plain SUMO increased. This is because of the combination of plain and spiral wires utilized in Secura cable that affects it bonding strength.
6.8. Load transfer performance of indented TG cable bolt

Test 11 and Test 12 show the loading performance of the indented TG cable, manufactured by Jennmar with and without application of pre-tension on the cable.

Test 11, Indented TG with zero pretension, gained the peak shear load of 31.09 t with a corresponding displacement of 33.2 mm. Test 12, Indented TG with applied 15 t pretension on the sample, appeared to have a peak shear load of 36.32 t with associated displacement of 30.87 mm. The shear load-displacement graph indicates the performance of such cables with zero and 15 t applied pretension (Figure 6-15).

As the graph shows (Figure 6-15) Test 11 failed prematurely at point A and B. Because of this premature failure on a wire strand, the peak shear load and associated displacement decreased. One possibility for this early failure could be for the speed rate of the test which was conducted in 1 min 43 seconds which resulted in an average rate of loading of 15 mm/min which is in excess of the limit of loading bound of 1 mm/min as per BS7861-2:2009 and ASTM standards. This stressed the problem of the manually applied load on the hydraulic pump which did not allow for a constant rate of loading to be applied. Each sample the rate of loading was found and this methodology flaw is discussed in detail in Section 6.18. and Table 6-3.
Due to an early failure of the cable in shear load, the effect of pre-tension for this type of cable, Indented TG, remains questionable. The load-displacement graph for Test 12 seems to be unaffected by the shock force as the rate of loading of 15 mm/min was significantly lower in comparison with that of Test 11. According to conducted experiments as well as previous studies, imposing pretension load on cable bolts would affect the peak shear load negatively and this is because of increasing the stiffness on the sample. However, this hypothesis did not agree with the Indented TG sample when pretension load was applied. Another test should be performed on Indented TG with zero tone pretension loads with recommended speed of loading to validate the hypothesis.
6.9. Load transfer performance of Plain SUMO cable bolt

Test 13 and Test 14 illustrate the loading performance of the plain SUMO cable, manufactured by Jennmar with and without application of pre-tension on the cable. The UTS for this type of cable, which has a diameter of 28mm is 63t including 6 bulbs. The shear load-displacement graph illustrates the performance of such cables with zero and 15t applied pretension.

The peak shear load gained by 0 pretensioned plain SUMO is 54.7t with a relative displacement of 71.8mm, whilst the 15t pretensioned Plain SUMO reached the peak shear load of 67.1t with a corresponding displacement of 78.2mm. Unlike the other tests, the inverse relationship between pretension and peak shear load was not maintained as by increasing pretension load from 0 to 15 tonne, the peak shear load increased accordingly. However, the increased peak shear load to 67.1 tonne is beyond its UTS and the reason is not clear.

![Figure 6-17: Plain SUMO test results](image-url)
The increase of pretension load was conducive to an improvement of peak shear load by 22.6% along with a 9% increasing in an associated displacement.

6.10. Effect of indentation on the loading performance of cable

SUMO and TG results enable us to examine the effect of indentation on the loading performance. As Figure 6-19 shows, the indented samples (Tests 9, 10, 11 and 12) gained the lowest peak shear loads amongst the all tested samples. A stress concentration is placed on cable wire when indentation is formed by machinery equipment through stamping process. Therefore, such stress is agreed through previous studies that has a detrimental effect on a strand shear capacity along with lesser cross-sectional area (Aziz et al., 2016b, Li et al., 2015).
6.11. **Load transfer performance of Superstrand cable bolt**

Test 15 indicates the loading performance of the plain Superstrand cable, manufactured by Jennmar with an application of pre-tension on the cable, 15 tonne. The UTS for this type of cable, which has a diameter of 21.8mm and consists of 19 plain wires is 60t with no bulb. The shear load-displacement graph illustrates the performance of such cable in appendix C.

The maximum shear load attained through this test for such cable was 51.4t with an associated displacement of 90.2mm. the procedure of the test clearly indicates that the sample underwent complete debonding. The principal reason for the sample debonded was the plain configuration of the strand, which was plain and as it has been previously mentioned, debonding occurs due to the surface roughness deficiencies of the cable. The sample clearly indicates that the only 85.7% of its UTS reached by shear load capacity. It is worth mentioning that the debonding affects the result adversely.

As it is shown in appendix B, the majority of wires failed in combination of tension as well as shear. More samples are required to be assessed in order to validate data and make comparison between them under various pretension conditions.

6.12. **Load transfer performance of Garford cable bolt**

Test 16 presents the loading performance of the Garford cable, manufactured by Jennmar without application of pre-tension on the cable. Since this type of cable is twin, therefore, all the mechanical components of this cable are doubled. The UTS for this type of cable is considered as 2×26.5t with a diameter of 2×15.2mm. The shear load-displacement of this cable is provided in appendix C.

The peak shear load was measured as 43.7t with a corresponding displacement of 46.8mm. During this test the cable underwent debonding and this could be predicted since the all wires were plain. Furthermore, the 80.9% of its UTS was reached through shear load. As it can be seen from appendix B, the majority of wires failed in combination of shear and tension.
6.13. **Double shear test analysis**

6.13.1. *Load transfer performance of Plain SUMO*

Test 1 and 2 explored the load performance of plain SUMO made by Jennmar, under zero and 15t applied pretension load. The plain 28mm SUMO has UTS of 65t. The plain SUMO sample results are shown on a load-displacement graph in Figure 6-20.

Test 1 was carried out on Plain SUMO with zero pre-tension, the test recorded the maximum shear load of 88.59t with an associated displacement of 100mm. Test 2 a 15t pre-tension load applied to the cable and the maximum shear load of 85.2t achieved with a significantly lower displacement of 88.2mm. The maximum axial load developed as a result of shear load for Test 1 and 2 recorded as 43.71t and 44.6t respectively (appendix C).

The plain SUMO cable agrees with the inverse relationship of the applied pretension load with shear resisted, meaning that with increasing the axial load from zero to 15 tonne is conducive to the peak shear load decrease. Test 2 with pretension to the system reduced displacement by 11.8% compared to zero pretension cable. However, an only 3.97% change observed by applying pretension load to the system.

![Shear Load vs Displacement Graph](image.png)

**Figure 6-20: Plain SUMO test results**

Point A on the shear load- displacement graph (Figure 6-20) illustrates the possible initial deformation of the central cable’s hollow core tube as well as concrete and grout
crushing at the shear zones. It is noted that the plain SUMO with no pretension did not undergo complete failure over 100mm allowable displacement. However, after completion the test, the sample was dismantled in order to gain a better understanding of the failure status of the wires, generating the schematic shown in Figure 6-21. As it is clear from the Figure 6-21, only 67% strands underwent failure mode along with the central core and 3 strands did not reach their UTS and remain working. However, the presence of a “cup and cone” among the snapped wires clearly shows that the strand failed in tension as an axial load has elongated the cable until the UTS of the cable was exceeded. In addition, no failure was detected on the other side of the central block (Left side) on either sample.
It is worth mentioning that each shear face of the sample was painted in a special pattern in order to examine if any friction would occur during the shear test. As a matter of fact, no friction was detected as all pattern remained intact and this can prove that all the data derived from double shearing test are pure in validity to examine the strength of the cable in shear. Moreover, the extent of concrete crushing, moving inwards towards the centre of the middle block for the sample with zero and 15t pre-tension was around 100mm (Figure 6-21).

6.13.2. Load transfer performance of Spiral SUMO

Test 3 and 4 explored the load performance of spiral SUMO made by Jennmar, under applied zero and 15t pretension load. The spiral 28mm SUMO has UTS of 63t. The spiral SUMO sample results are indicated in load-displacement graphs in Figure 6-22, Figure 6-23, Figure 6-24 and Figure 6-26.
Test 3 was conducted on spiral SUMO with zero pre-tension recorded the maximum shear load of 81.49t with an associated displacement of 93.43mm. Test 4 a 15t pretension load applied to the cable and the maximum shear load of 76.7t and corresponding shear displacement of 85.74mm was reached. The axial load developed as a result of vertical shear displacement on the cable bolt for test 3 and 4 was 36.68t and 39.93t respectively. Figure 6-22 and Figure 6-24 illustrates all the results derived from the tests.

![Graph](image)

**Figure 6-22: Test 3 Spiral SUMO- o pretension**

As it can be seen from the Figure 6-26 beyond the vertical displacement of each test 93.43mm and 85.74mm, various shear load drops occurred in which as a result of strand failure. The larger shear load drop can be likely due to failure of some wires simultaneously and relatively smaller shear load drops are due to single strand failure. Figure 6-22 illustrates the shear load and axial load developed in the spiral SUMO without application of pre-tensioning. The point A is the place that barrel and wedge settlement-adjustment occurred and it seems a crushing happened in concrete or in the grout interface as the drop is noticeable. Point B is where the peak shear load of the sample occurred and it is the place that likely 1 or 2 strands failed on the side of the load cell 1. This can be evident from the axial load readings on point B which have not a
significant drop on its axial load, from 36.125t to 31.588t. Point C shows when the major failure occurred including the central core failure, although the central core is not a load-bearing element. Point I indicates the failure of only one strand on the other side of the shear face of the central block (Left side) as it is shown in Figure 6-23. However, the major failure occurred on the right side and only 67% of the strands failed.

![Figure 6-23: Strand failure and concrete crushed, test 3](image)

As it can be seen from the Figure 6-23 the concrete crushing extension length can prove that the failure occurred on the right side as the length of the crush is less than the other side which means that the number of snapped wires on the right side is more than the other side.

![Figure 6-24: Test 4 Spiral SUMO- 15t pretention](image)
Figure 6-24 indicates that failure occurred at point A. Due to the large drop, it seems that the majority of the cable failed in combination of shear and tensile strength and after the strand lose its capacity other wire starts snapping. The sequence of each snap can clearly be seen from the Figure 6-25. It is noted that the central hollow-groutable core is not considered as the load-bearing element and all the outer elements (9 wires) carry the load (Craig and Aziz, 2010b). Point 1 and 2 shows the failure of the strand elements and as it failed at the same time then it proves the possibility that the failure of wires occurred at either side of the central block. All the strands failed on the right side and only one strand snapped on the left side. The extension crushing length is 75mm.

![Image of failure and concrete crushed](image1)

![Image of extension length](image2)

Figure 6-25: Strand failure and concrete crushed, test 4

The spiral SUMO cable agrees with the inverse relationship of the applied pretension load with shear resisted as similar with plain SUMO. Test 4 with pretension to the system reduced displacement by approximately 9% compared to zero pretension cable. However, an only 6.2% change observed by applying pretension load to the system.
6.13.3. Load transfer performance of Plain MW10

Test 5 and 6 illustrates the load transfer performance of the plain MW10 with different pretension values manufactured by Megabolt. This type of cable consists of a king wire covered by 10 plain wires. The UTS of 70t is allocated for this kind of cable which is the highest among the cables tested and also MW10 has one more wire compared to other cables tested, so the other cables have 9 wires. The MW10 sample results are depicted on a load-displacement graph in Figure 6-28.

Test 5 was conducted on Plain MW10 with zero pre-tension recorded the maximum shear load of 83.79t with an associated displacement of 100mm, however, it is noted that no strand failure detected in 100mm so it was decided to continue testing for 20mm more that suddenly failure was occurred in 105mm with a peak shear load of 87.88t, even though the failure is due to concrete smashing as no strand failure detected, as it is shown in Figure 6-27. Moreover, the bonding agent which was grout did not flow into the annulus to anchor the strands to the concrete as the whole was not open at the end. This could be a result of smashing concrete and not snapping any strand. In other word, the load did not transfer correctly which reinforced the importance role of grout.
pretension load applied to the cable and the maximum shear load of 92.31t and corresponding shear displacement of 88.5mm are obtained.

Figure 6-27: Concrete failure, test 5

This type of cable disagrees with the idea that increasing pretension load will decrease the peak shear load but shear displacement. The data suggest that increasing the
pretension load not only increase the peak shear load by approximately 5% but also reduces the shear displacement by 8.96%.

Point A on the plain MW10 with 15t pre-tensioned shear load- displacement graph is the typical barrel and wedge adjustment as well as concrete crushing during the shearing process. This occurred at 22.5t of vertical shear load and the vertical shear displacement of 32mm. Point 1 shows the simultaneous failure on either side. As it is depicted in Figure 6-29 only 5 strands failed on the right side and one snapped occurred on the left side. The extension crushing length for the right side of the block is detected as 100mm.
6.13.4. Load transfer performance of Spiral MW9

Test 7, 8 and 9 illustrates the load transfer performance of the spiral MW9 manufactured by Megabolt. The UTS of 62t is allocated for the spiral strand with an elongation at strand failure of 5- 6%. The tests conducted on spiral cables with zero, 7.5 and 15t pretension values. Figure 6-34 depicts the all load and displacement data for the spiral MW9 tests.

Test 7 explored the transferring load by without application of pre-tensioning. This test did not undergo any failure as depicted in Figure 6-31. The maximum shear load was reached is 84.76t in 100mm (point A). As the sample did not break at the 100mm, it was decided to continue testing for 20mm more (Point C) in order to investigate whether the cable would fail. The Figure 6-31 shows that the sample did not undergo any failure even beyond the restricted displacement (120mm) and the noticeable drop (Point B) is only due to the concrete failure, this can be evident from the heavily crushed zone in the vicinity of the shear face generated in Figure 6-30. The extension crushing length was approximately 150mm. The maximum shear load in this case was 93.86t in 115.21mm. The maximum axial load development for test 7, 8 and 9 as a result of shear load is 36.3t, 45.91t and 44.09t.

Figure 6-30: Concrete crushed and no strand failure, test 7
By increasing the initial axial load from zero to 7.5t and 15t can clearly see that there is a reverse relationship occurred between pretension and shear load resisted. Test 8 explored the peak shear load of 90.7t with shear displacement of 89.7mm and Test 9 investigated the maximum shear load of 83.7t with corresponding shear displacement of 88.47mm with the application of 7.5t and 15t pre-tensioning. As it can be seen from the Figure 6-32, there is a huge difference in shear displacement between samples with pretension load and without pretension load. The differences are 27.6% and 29.4% respectively. However, the change in peak shear load is not significant among samples.
Figure 6-32 demonstrates that Test 7, 8 and 9 have the similar immediate stiffness on the load-displacement graph. As previously discussed, by applying pre-tension the stiffness in the system is affected enormously and therefore the elastic modulus is decreased.

Increasing the pre-tension load is hypothesis that decreases the peak shear load due to reduced flexibility of the cable. This hypothesis was correct for spiral MW9. Moreover, data suggests a nearly identical elastic region displacement for all Tests 7, 8 and 9 where Test 7 should have a greater shear displacement in the elastic zone. It is noted that the stiffness in Test 7 is because of brittle host material during initial loading which affects the results.

Comparing Test 8 and 9 there is only little benefit to increase pre-tensioning load from 7.5t to 15t when displacement is important. Increasing the pre-tension load caused to 1.35% and 8.37% decrease in shear displacement and peak shear load respectively.

In overall look by considering Figure 6-33 can be concluded that when direct shear load is applied the loading performance of the cable is influenced significantly by the application of pretension. The peak shear load was influenced adversely in plain and indented SUMO as well as spiral MW9 as a decreasing trend is represented by the Figure 6-33. It is worth mentioning that although Spiral MW9 with non-pretension load did not break at 100mm, it means that the ultimate capacity of the cable was not reached. This test was continued for more 20mm displacement and the final capacity was reached up to 93.89t, then it can be pointed out that the peak shear load decreases by increasing pre-tension load.

![Figure 6-33: Influence of cable strand pretension on the joint shear strength](image)

On the other hand, Plain MW10 refute the hypothesis that pre-tension load decreases the peak shear load. This is because the application of pre-tensioning had a reverse impact on the cable although its related-displacement reduced.
Pre-tension reduced peak shear load on average by 5.5% across the comparable pre-tensioned strands and it can be considered that the pre-tensioned sample achieved approximately 90 to 95% of the typical load resisted by non-pre-tensioned equivalent cable. Interestingly, plain MW10 reversely reached only 90% of its ultimate capacity when it was pre-tensioned. Moreover, it is concluded that the extension crushing length has a direct correlation with the peak shear load as well as pre-tension load, as data suggested that by increasing pre-tension load and decreasing peak shear load the length of the crushing concrete reduced.

Figure 6-34 clearly indicates that applying pre-tension load on the cable reduces displacement on average by approximately 10% across the comparable pre-tensioned strands. Although displacement for Spiral MW9 with the application of pre-tensioning from 7.5t to 15t is negligible and as small as 1.4% change, however, this change can be significant when the pre-tensioned load changed from zero to 7.5t or even 15t, the shear displacement reduced by 27.6% or 32.8% respectively. Moreover, plain MW10 did break beyond the restricted displacement, as spiral MW9, and without considering the limitation for the vertical movement of the central block, this reduction could be increased by 18.6% whilst the displacement of such sample with considering the
limitation is only 13% and the 5.8% difference occurred when the cable reached its ultimate capacity and did not reached ultimate capacity.

6.15. **Comparison of the plain and spiral cable geometry on the loading performance of cables**

The effect of indentation as well as smooth surface of the cable was examined through the SUMO, MW9 and MW10 cables. Figure 6-33 illustrates that the smooth samples (Test 1, 2, 5 and 6) gained the greater peak shear loads for all the cables tested. The reason of this greatness seen on smooth surface as previously discussed is because of the stress concentration applied to the wire strands through the stamping action when the indentation are formed, which primarily leads to a mass loss and creation a strand with lesser cross-sectional area, which ultimately make plain cables have higher peak shear load.

Furthermore, the effect of indentation can be analysed through comparing the plain and spiral SUMO as well as plain MW10 with spiral MW9 test results. Therefore, the following tests are considered to make a comparison:

- Test 1 and Test 3
- Test 2 and Test 4

Test 1 and Test 3 utilized non-pretension load to the SUMO samples. Plain SUMO obtained a maximum shear load 8.71% higher than the indented SUMO counterpart. However, spiral SUMO was beneficial for lower displacement, reducing the displacement at peak by 7.1% from 100mm in smooth SUMO to 93.43mm in the spiral SUMO sample.

Test 2 and Test 4 involved with 15t the pretension load on the SUMO samples. Plain SUMO obtained a maximum shear load 11.1% higher than the indented SUMO counterpart. However, spiral SUMO was advantageous for lower displacement, reducing the displacement at peak by 3% from 88.2mm in smooth SUMO to 85.4mm in the spiral SUMO sample, which is not significant.
6.16. **Reaction forces during the shearing**

According to Jalalifar (2006), in order to analyse the reaction behaviour of a cable bolt, there is an inevitable assumption that should be considered.

1. When a cable bolt is subjected to shearing, it is deformed and this deformation will lead to moving a shear force and normal force in the system.
2. With increasing the load, its longitudinal axis is deformed into a curve which will cause to generating two components; lateral load (Q) and axial load (N) and also two critical points will appear and require to be studied; one on rock joint intersection with maximum shear stress and no bending moment and the other one in maximum bending moment, which is called hinge point, with no shear stress.

Based on the theory of the beam, the axial stress produces a uniform stress distribution and the bending moment generates a linearly varying stress as it is shown in Figure 6-35. According to this theory the deformation of the cable bolt after applying load can be analysed.

![Figure 6-35: The load generation along the cable bolt subjected to shearing](image)

6.17. **Cable bolt behaviour**

According to appendix C, all graphs show the behaviour of the different cable bolts under shearing load. The graph divided into three phases; Elastic stage, non-linear stage (elastoplastic) and Plastic stage.

6.17.1. **Elastic stage**

This stage is associated with the elastic behaviour of the cable. Meaning that no deformation is appeared during this stage and the behaviour of the cable is under
control. At this point by removing the load, the cable is back into its original shape. This linear section of the graph is characterized with a rapid increase of the shear load at a relatively small displacement. In this section some small fractures occurred, however, they do not have an impact on the performance of the system and also barrel and wedge adjustment settled at this section. Furthermore, this linear section shows the stiffness of the system which is changeable by the effect of the application of the pre-tension load. More initial axial load, steeper the curve as well as shorter the elastic stage.

6.17.2. Plastic stage
In this stage there are two stages, elastoplastic stage and plastic stage. Elastoplastic stage is known as a transition stage from elastic to plastic stage. When the cable reached its yield point, any more loads beyond this stage will lead to creation a permanent deformation on the cable and it is known as plastic deformation. In this stage the displacement increased slowly with increasing load. After the cable reached its ultimate tensile strength, it would enter its plastic stage which means that any more loads beyond its ultimate tensile strength will absolutely lead to failure of the system.

6.17.3. Axial load development
With considering the development of axial load in all DST samples, it can be clearly seen that there are three distinguishable phases as shear load-displacement graphs. The first phase is the slow loading, the second is rapid loading and the third is step drop loading. As it is obvious from the all figures, the axial load is developed slowly by increasing the applied load in the first phase of the graph, especially the cable without initial axial load and this is because of the high elastic modulus of the cable and having greater elastic region. However, once the cable bolt is pretensioned, the stiffness starts increasing in the system which causes a reduction to elastic modulus as well as shortening the elastic region and eventually the axial load development would occur faster in compare to a cable without the application of pretension load. As the load is increasing, the axial load development enters in its second phase which is the plastic stage and the load development occurred swiftly with associated displacement until the cable reaches its ultimate strength.

According to Figure 6-35, the produced axial load in a cable bolt is as a result of shear force as well as pretensioning. Thus, it can be mentioned that the contribution of the cable bolt is as a result of axial load as well as shear load, which the axial load is a
summation of axial load due to pretensioning along the cable and axial load developed due to shear displacement. Therefore, the reason why the maximum axial load is higher in samples, which have an initial axial load, is due to the contribution of pretension load in the system, this can be reinforced from the laboratory investigations and also Jalalifar (2006) noted that a bolt can be pulled along its axis without pretensioning or end plate subjected to shearing, however with including pretensioning load or attaching an end plate, tensile stresses are produced along the bolt and the bolt contribution will increase.

6.18. **Single and double shear test comparison**

An important factor that needs to be taken into account is the rate of shearing load applied in both tests, which were different. The rate of loading was higher in single shear test in comparison with double shearing test as the double shear test was adopted the rate which was recommended by BS 7861-2 (British Standard, 2009) and F 432-04 (ASTM, 2005). This might influence the validity and credibility of the results. These rates were different for all samples examined in single shear tests from 1.6 to 38mm/min as shown in Table 6-3.
<table>
<thead>
<tr>
<th>Cable type</th>
<th>Peak shear load (t)</th>
<th>Displacement (mm)</th>
<th>Rate of loading (mm/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MW9- Spiral- 6 bulbs - 0t pretension</td>
<td>47.73</td>
<td>43.5</td>
<td>23</td>
</tr>
<tr>
<td>MW9 - Spiral – 6 bulbs- 15t pretension</td>
<td>52.6</td>
<td>41.4</td>
<td>26</td>
</tr>
<tr>
<td>MW10 - Plain- 6 bulbs - 0t pretension</td>
<td>63.84</td>
<td>62.57</td>
<td>38</td>
</tr>
<tr>
<td>MW9 – Spiral – No bulb - 15t pretension</td>
<td>49.7</td>
<td>41.73</td>
<td>11</td>
</tr>
<tr>
<td>MW10 - Plain-6 bulbs- 15t pretension</td>
<td>60.39</td>
<td>56</td>
<td>17</td>
</tr>
<tr>
<td>MW10 - Plain-0 bulb - 15t pretension</td>
<td>68.34</td>
<td>68.24</td>
<td>23</td>
</tr>
<tr>
<td>Secura - 0t pretension, 6 bulbs</td>
<td>64.69</td>
<td>51.8</td>
<td>11</td>
</tr>
<tr>
<td>Secura - 15t pretension, 6 bulbs</td>
<td>55.9</td>
<td>45.9</td>
<td>12</td>
</tr>
<tr>
<td>ID-SUMO - 0t pretension, 6 bulbs</td>
<td>40.43</td>
<td>40.91</td>
<td>20</td>
</tr>
<tr>
<td>ID-SUMO - 15t pretension, 6 bulbs</td>
<td>37.36</td>
<td>30.9</td>
<td>13</td>
</tr>
<tr>
<td>Indented TG - 0t pretension, No bulb</td>
<td>31.09</td>
<td>33.2</td>
<td>21</td>
</tr>
<tr>
<td>Indented TG - 15t pretension, No bulb</td>
<td>36.32</td>
<td>30.87</td>
<td>15</td>
</tr>
<tr>
<td>Plain SUMO - 0t pretension, 6 bulbs</td>
<td>54.7</td>
<td>71.8</td>
<td>3.1</td>
</tr>
<tr>
<td>Plain SUMO- 15t pretension, 6 bulbs</td>
<td>67.1</td>
<td>78.2</td>
<td>4</td>
</tr>
<tr>
<td>Superstrand – 15t pretension</td>
<td>51.4</td>
<td>90.2</td>
<td>1.6</td>
</tr>
<tr>
<td>Garfoprd – 0t pretension</td>
<td>43.7</td>
<td>46.8</td>
<td>2.2</td>
</tr>
</tbody>
</table>

Controlling the rate of loading is significantly important as the excessive speed of loading from the over acceleration of the hydraulic pressure cause to impose a shock load to the system which affects the results adversely. However, regarding this matter, the rate of loading suggested by standard is 1mm/min in order to eliminate the effect of shock loading on the system. There is a feature in concrete based material which is
widely known as creep. This characteristic of concrete refers to a deformation of concrete under sustain loads over time. However, this element can be affected by the rate of loading in experimental studies. The faster loading imposes to the system, it is more likely to be removed the material creep from the results. As it is shown in Table 6-3 each test sample was loaded through a higher rate than recommended by the standard, which in turn, the results of the single shear tests were not only influenced by the shock loading but also the effect of creep is removed, especially for the test one to 12. This is the principal reason for the significant discrepancy between the results of single and double shear tests in peak shear load. Hence, for the last four tested samples, test 13, 14, 15 and 16, the aim of the test was to control the rate of loading and attempt to keep the rate as close as possible to 1mm/min. Therefore, this imperfection of single shear apparatus reinforces the fact that the manual application of pressure should be replaced by an automatic device for future investigations in order to achieve a more accurate and appropriate results. However, it is worth mentioning that the rate of loading for double shear method is set to 1mm/min as recommended by the standard. Furthermore, radial confinement was provided by the single shear apparatus through utilizing steel clamp. The effect of confinement in previous studies has been indicative of an increasing the bond strength between grout-cable interface (Hyett et al., 1992). However, this parameter is not provided by double shear apparatus due to its rectangular shape. Consequently, it is recommended that the influence of confinement be considered for the next generation of the MKIII in the future to provide a more credible results, in particular, when the results of double shear test is going to be compared to single shear results. By adding such characteristic to the apparatus the natural environment behaviour of the field is more appropriately replicated as this pressure is provided through the adjacent of the strata to the supporting system.

From this study, it is significant to note that the effect of debonding is possible to be examined through the single shear apparatus, whilst double shear apparatus has deficiency to provide such behaviour. However, the advantage of double shear apparatus enables to monitor the performance of axial load propagates through the system, while this cannot be carried out by single shear apparatus.

The other important factor that makes difference between these two methodologies is the expense and the money incurred from the tests. Setting up the single shear test is not
only requires an expert supervision but also the testing time as well as dismantling samples are time consuming. The new generation of double shear apparatus utilized at the University of Wollongong which is frictionless allows for direct comparison to the single shear results. However, the results of the single shear test are currently limited as testing is still in progress. Moreover, the units of the results are completely different as single shear test peak loads are in Tone and double shear test results are in KN, so in order to compare the results all the double shear tests peak load were converted from KN to t and also divided by two as the double shear test reported peak load is the peak load of two faces. Table 6-4 illustrates the single and double shear values for the samples which were examined in both methods.
The above Table 6-4 and the Figure 6-36, clearly show that the ID-SUMO sample with 15t pretension load is similar across the tests. However, the results for the MW9, MW10, and Plain SUMO as well as ID-SUMO- 0 pretensions are different. Although ID-SUMO, plain SUMO- 0t pretension and MW9 cables have close results by comparing single shear and double shear test results. This difference is due to the rate
of loading as in single shear test imposes an element of shock loading on the system. However, the impact of rate of loading is minimal and it suggests that for these types of cables the rate of loading is not influential. Moreover, a considerable difference can be seen from the MW10 samples. The peak shear load in single shear test is approximately 50% greater than the maximum load resultant from double shear test. This difference is due to the fact that the sample was experienced full debonding during the single shearing test. For eliminating this huge difference to gain the similar results as previously mentioned the embedment length of the single shear apparatus should increase. Furthermore, Plain SUMO-15 t pretension has 35% difference with comparing the results in single and double shear tests. This huge difference is as a result of using bulb structure in single shear test whilst no bulb was utilized on cable in double shear test. This reinforces the effect of bulbed structure on plain wire strands. The confinement pressure is another element that affected the differentiation of the results between single and double shear tests, as it was mentioned above, the newly modified double shear apparatus has deficiency of imposing confinement pressure throughout the testing and that has caused to a premature failure occurred on the cables tested by double shear apparatus as confinement pressure increases the bonding strength of the system which would end up leading to an improvement of peak shear load.

Figure 6-36: Comparison of Single and Double shear test results
6.19. **Chapter summary**

In this chapter, the peak shear load and associated shear displacement for all 16 samples in single shear tests and 9 samples in double shear test was examined and analysed. The data reported from the single shear test suggests that the plain MW10 is superior only in terms of shear load strength, however the embedment length was insufficient to utilize this type of cable as it was debonded during the test. Moreover, data suggest that the UTS of the strand was affected adversely by the effect of indentation created on the surface of spiral cable bolts which results in an appearing a lower shear strength of the system. Reversely, in such cables the shear displacement performed appropriately due to its surface roughness, which provides a stronger bond between grout and cable interface.

The results of the single and double shear test was compared and it was concluded that due to varying on the rate of shearing load applied, which was the reason of creation of shock loading, the peak shear load was different when the same samples were compared. Furthermore, because double shear apparatus has rectangular shape, the effect of confinement is unable to be provided. Consequently, it is recommended that the influence of this pressure consider for the next generation of the MKIII in the future to provide a more credible results, in particular, when the results of double shear test is going to be compared to single shear results.
Chapter 7: CONCLUSIONS AND RECOMMENDATIONS

The previous studies was reviewed during the course of this research regarding pull testing, single shearing tests as well as double shearing test. In this study a new single shear methodology is utilized in order to address the shortages demonstrated by the current single shearing standard (BS7861-2:2009). This new single shear methodology enables to examine the extent of debonding occurs in plain or smooth wire strands. However, this new apparatus still has deficiency to provide the value of the axial load imposed throughout the test.

The new double shear apparatus was utilized in order to eliminate the effect of friction between the shear-joint faces. Furthermore, the deficiency of providing axial load imposed to the system in single shear test was eliminated by utilizing double shear test. Therefore, the type of apparatus as well as the single or double shearing method being used to examine the shearing capacity of cable bolts is reliant on the objective of the research whether the effect of debonding requires to be examined or the increase the axial load needs to be monitored.

A logical comparison was made in order to compare the performance of both methods to explore the credibility and validity of data. Data suggested that cable bolt profile configuration is a significant parameter in load transfer capacity of cable bolts. From the Single shear tests:

Naturally, the spiral cable creates a greater bonding strength at the cable-grout interface due to its roughness surface. Therefore, in case of spiral cables, the additional bulbs did not affect the load performance in comparison with plain MW10 cables through comparing the displacement. Moreover, in average, the plain MW10 achieved 90% of its UTS while spiral MW9 achieved 75% of its UTS. However, it is worth pointing out that the results for MW10 has been affected by debonding.

All plain MW10 cables was undergone debonding this proposed that the embedment length of the anchor cylinder for the test was not adequate for the plain strand wires.

According to Figure 6-9 among the all six tests Plain MW10 proved that it has a potential to resist higher shear load before failure, whilst the Spiral MW9 samples offered a lower shear displacement. Consequently, depending on the conditions of the
strata if the minimal movement is concerned so according to data spiral MW9 would be more appropriate option and if higher peak shear load is required, plain MW10 would therefore be more efficient.

Bulb structures was shown to be more effective on plain wires rather than spiral wires as the latter cables presented almost no change on the loading performance. Furthermore, the application of bulbed structure on Plain MW10 affected the peak shear load as well as shear displacement adversely. This is due to a rigid structure of that the bulb provides.

The performance of Secura HGC in shear load is similar with plain MW10 cable. It is understandable from comparison of Secura and MW10 to MW9 that the existence of plain wires in the strand affects the UTS of the strand enormously which leads to an increasing the peak shear load of the cable. It is significant to point out that the Secura HGC did not undergo any debonding and this is because of irregularity of the cable strand surfaces which is a combination of plain and spiral. The study suggests that the combination of the two profiles, plain and spiral, is advantageous in terms of excluding debonding without adversely affect the strength of the cable.

As data suggests that by placing pretension load on SUMO cables the reliability and integrity of the excavation increases. By comparing Plain Sumo cables to Secura cable bolts, imposing pretension load can be mentioned that has a detrimental effect to the reliability of the Secura cable bolt as the peak shear load decreased. However, shear displacement for Secura cable reduced whilst the corresponding shear displacement for plain SUMO increased. This is because of the combination of plain and spiral wires utilized in Secura cable that affects it bonding strength.

Due to an early failure of the TG cable type in shear load, the effect of pre-tension for this type of cable, Indented TG, remains questionable.

SUMO and TG results enable us to examine the effect of indentation on the loading performance. As Figure 6-19 shows, the lowest peak shear loads for all the cables tested was gained by the indented samples (Tests 9, 10, 11 and 12). Previous research agree to the detrimental effect of indentation on a strands shear capacity which is believed to be caused by the stress concentrations applied to the wire strands through the stamping
action when the indentions are formed, in combination with the lesser cross-sectional area (Li et al., 2015, Aziz et al., 2016b).

From the double shear tests:

The newly modified apparatus, MKIII, has shown that the results are more consistent with comparison to the previous studies as the additional braces on each side of the apparatus prevents friction between the shear faces throughout a test. However, the concrete crushing due to creation of hinge point as a result of concrete deformation on jointed areas are excessive and this should be taken into account as a deficiency of such new generation of double shear apparatus currently utilize at University of Wollongong and needs to be addressed.

It is worth pointing out that plain SUMO, plain MW10 and spiral MW9 with zero tonne pretension load did not undergo failure in 100mm shear displacement, therefore, this made the comparison of peak shear load with corresponding shear displacement among the samples to be difficult.

The effect of the pretension on all samples agreed with the hypothesis that the increasing the pretension load affects the peak shear load adversely, however, Plain MW10 refuted this theory and this is because did not break at 100mm displacement. It is recommended that more examination be conducted on MW10 cable bolts to clear the results. Moreover, by increasing pretension load, shear displacement at peak shear load decreased and this agreed with the hypothesis.

Cable bolt deflection for all samples was detected a value between 75 to 150mm, with an average of 100mm which is three times higher than diameter of cables

Among the all cables tested, plain MW10 with applied 15 tonne pretension load has the highest peak shear load and Spiral SUMO with 15 tonne pretension load has the lowest shear displacement. The higher shear strength of MW10 is due to an additional wire strand and having higher UTS compared to other samples.

Plain SUMO with zero pretension proved that all the data derived from double shearing test are pure in validity to examine the strength of the cable in shear as there was no
friction detected from the sample. Plain Sumo reached in average 67% of its Ultimate Tensile Strength, whilst Indented SUMO attained only 63% of it UTS.

From the comparison of two methodologies:

A considerable difference could be seen from the MW10 samples. The peak shear load in single shear test was approximately 50% greater than the maximum load resultant from double shear test. This difference reinforced the fact that the sample was experienced full debonding during the single shearing test and also stress that the embedment length was insufficient.

The rate of shearing load applied in both tests were different and the rate of loading was higher in single shear test in comparison with double shearing test as the double shear test was adopted the rate which was recommended by the British Standards (BS7862-2) and ASTM (F-432). This might influence the validity and credibility of the results. These rates were different for all samples examined in single shear tests from 9 to 38mm/min (Test 1 to 12). This rate difference across all samples in single shear tests led to all samples experienced an extra load as it was called shock load. This shock load was recognized as the peak shear load difference between the same samples tested via different methodologies.

The single shear apparatus provides radial confinement to the sample through the use of steel clamps. Previous studies by Hyett et al. (1992) have shown that confinement will influence on the results gained as increasing confinement will improve bond strength. The double shear test does not allow for the confinement of the sample due to the rectangular shape of the apparatus. It is recommended that the next generation of the double shear test to examine the impact of confinement to create more reliable results when comparing the results to different apparatuses. This improvement to the double shear test will further improve the replication of in situ failure as adjacent rock strata will provide this force to the ground support system.

The other important factor that makes difference between these two methodologies is the expense and the money incurred from the tests. Setting up the single shear test is not only requires an expert supervision but also the testing time as well as dismantling samples are time consuming and requires heavy physical work.
REFERENCE LIST


CRAIG, P. & AZIZ, N. I. 2010b. Shear testing of 28 mm hollow strand” TG” cable bolt.


FULLER, P. & COX, R. 1975. Mechanics of load transfer from steel tendons to cement based grout, Division of Applied Geomechanics, CSIRO.


HAWKINS, E. B. 2016. SHEAR TESTING OF CABLE BOLTS USING AN INTEGRATED SINGLE SHEAR TESTING APPARATUS. University of Wollongong.


JALALIFAR, H. 2006. A new approach in determining the load transfer mechanism in fully grouted bolts.


MAHONY, L. & HAGAN, P. 2006. A laboratory facility to study the behaviour of reinforced elements subjected to shear.


RAJAIE, H. 1990. Experimental and numerical investigations of cable bolt support systems.
SMITH. 2002. Load Transfer Mechanism in Bolts by Triaxial Test. Bachelor of ENginnering (Mining), University of Wollongong.
THOMAS, R. 2012. The load transfer properties of post-groutable cable bolts used in the Australian coal industry. 31st International Conference on Ground Control in Mining, 4, 1-10.

Appendix A

Following pictures are taken from all samples tested through single shear apparatus after failure.

Test 1 - Plain MW10, 15 tonnes pretension, No bulbs

Test 2 – Plain MW10, 0 tonne pretension, 6 bulbs

Test 3 – Plain MW10, 15 tonnes pretension, 6 bulbs
Test 4 – Spiral MW9, 0 tonne pretension, 6 bulbs

Test 5 - MW9, 15 tonnes pretension, 6 bulbs

Test 6 - MW9, 15 tonnes pretension, No bulbs
Test 7 – Secura HGC, 0 tonnes pretension

Test 8 – Secura HGC, 0 tonnes pretension

Test 9 – Indented SUMO, 0 tonne pretension
Test 10 – Indented SUMO, 15 tonne pretension

Test Eleven – Indented TG, 0 tonnes pretension

Test 12 Indented TG, 15 tonnes pretension
Test 13 – Plain- SUMO, 0 pretension, 6 bulbs

Test 14 – Plain- SUMO, 15 pretension, 6 bulbs

Test 15 – Plain Superstrand, 15 tonnes pretension, No bulbs
Test 16 – Plain Garford, 0 pretention, Bulbed
Appendix B

Test 1  Test 2  Test 3  
Test 4  Test 5  Test 6  
Test 7  Test 8  Test 9  
Test 10  Test 11  Test 12
Test 13

Test 14
Appendix C

Single Shear Results:

![Graphs showing single shear results](image)
Double Shear Results: