Shear behaviour of fully grouted bolts under constant normal stiffness condition

Ashitava Dey

University of Wollongong

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SHEAR BEHAVIOUR OF FULLY GROUTED BOLTS
UNDER CONSTANT NORMAL STIFFNESS CONDITION

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

DOCTOR OF PHILOSOPHY

from

UNIVERSITY OF WOLLONGONG

by

Ashitava Dey
B.E. (Cal), M.Tech (ISM), AMIE (India).

Faculty of Engineering
2001
I, Ashitava Dey, declare that this thesis, submitted in fulfilment of the requirements for the award of Doctor of Philosophy, in the Faculty of Engineering, University of Wollongong, is wholly my own work unless otherwise referenced or acknowledged. The document has not been submitted for qualifications at any other academic institution.

Ashitava Dey
3rd July 2001
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Three years in the making! A cast of thousands!
ABSTRACT

Research on rock bolting technology has been extensively pursued, in both civil and mining engineering, for several decades. The study of bolting technology embraced initially rigid re-bars and subsequently flexible cable bolts. Generally, the past studies were focused on bolt strength and their load bearing capacities in relation to physical size (length and diameter) and material properties. Very little work has been reported on the influence of bolt surface configuration on the load transfer mechanisms between the bolt and the host rock. Also, previous research on the influence of bolting on rock joint reinforcement and stabilisation has been carried out under Constant Normal Load (CNL) conditions, which does not reflect the actual deformation behaviour of reinforced joints. This matter has lately been recognised, and some recent publications have highlighted the shortcomings of the tests conducted under CNL conditions. Accordingly, this thesis is concerned with various aspects of bolt research related to the better understanding of the load transfer mechanisms between the bolt/resin/rock, and the impact of infill on the reinforced joint shear strength under Constant Normal Stiffness (CNS) condition. Two types of bolts, commonly used in Australian coal mines were used for both laboratory and field investigations, and referred to as type I and type II bolts, respectively.

The shear behaviour of bolt/resin interface was studied under CNS condition in the laboratory by testing cast resin/plaster samples in a specially constructed CNS apparatus. The samples were tested for various loading cycles at initial normal stress (\(\sigma_{no}\)) levels ranging from 0.1 to 7.5 MPa, representing the in-situ horizontal stress
condition in the field. Laboratory test results indicated that, bolts with deeper rib profile and wider rib spacing (e.g. bolt type I) offers higher shear resistance at low confining pressures less than 6 MPa, whereas, bolts with shallower rib profile with narrower rib spacing (e.g. bolt type II) offers marginally higher shear resistance at confining pressures exceeding 6 MPa. The maximum dilation of the bolt/resin interface occurs at a shear displacement of about 60% of the bolt rib spacing.

The laboratory study on the shear behaviour of the bolt/resin interface of fully grouted bolts was supported with field investigations in two local coal mines. At West Cliff and Tower Collieries, in the Southern Coalfield of Sydney Basin, NSW, Australia, 18 instrumented bolts and 6 extensometer probes were installed at three different sites. Field investigations in those mines revealed that, the load transfer on the bolt is influenced by; a) the confining stress condition in the field, b) the strata deformation, and c) the surface profile of the bolts. The influence of front abutment pressure was observed by sharply increasing load build up on the bolts, when the approaching longwall face was 60m and 150m from the test sites, in the travelling and belt roads, respectively. The field study also showed that, under the influence of low horizontal stress (both in magnitude and the direction), type I bolt offered significantly higher shear resistance, whereas under high influence of high horizontal stress, type II bolt offered marginally greater shear resistance at the bolt/resin interface, which were in accordance with the findings of laboratory testing.

The shear behaviour of bolted and non-bolted joints containing infill material, up to 7.5 mm in thickness, was studied under various initial normal stress levels between
0.13 MPa and 3.25 MPa, at a constant strain rate of 0.5 mm/min and a constant stiffness of 8.5 kN/mm. Significant reduction in shear strength was observed when the joint contained a layer of clay infill of 1.5 mm. Bolting contributed to increasing the strength and stiffness of the joint composite, except at large normal stress levels and at high infill thickness. The dilation and overall friction angle for bolted and non-bolted joints were also compared along with stress profiles. At high infill thickness (t/a>1), the shear behaviour under both CNL and CNS conditions was found to be similar for both bolted and non-bolted joints, while at low infill thickness (t/a<0.3), the CNL strength envelope plotted significantly above the CNS envelope. Thus, for rough joints with little or no infill, the CNS behaviour is more realistic in practice, especially for underground mining conditions in bedded or jointed rock.

An analytical model is presented using Fourier transform and the hyperbolic stress-strain simulation, to predict the shear behaviour of both clean and infilled bolted joints. The values of shear stress, normal stress, and dilation predicted by the analytical model were tallied favourably with the laboratory results. The UDEC model presented in the thesis underestimated the shear strength when compared with laboratory tested values, whereas it overestimated both the normal stress and dilation, especially at high initial normal stress conditions. Identical shear behaviour was observed, in both CNL and CNS UDEC model predictions, at low shear displacements and at high initial normal stress conditions, whereas the shear stress predictions were different at low normal stress conditions.
# CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TITLE PAGE</td>
<td>i</td>
</tr>
<tr>
<td>CERTIFICATION</td>
<td>ii</td>
</tr>
<tr>
<td>ACKNOWLEDGEMENTS</td>
<td>iii</td>
</tr>
<tr>
<td>ABSTRACT</td>
<td>v</td>
</tr>
<tr>
<td>CONTENTS</td>
<td>viii</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>xv</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>xxiv</td>
</tr>
<tr>
<td>LIST OF SYMBOLS AND ABBREVIATIONS</td>
<td>xxv</td>
</tr>
</tbody>
</table>

## Chapter 1. INTRODUCTION

1.1 General background  
1.2 Key objectives  
1.3 Outline of the thesis  

## Chapter 2. ROCK BOLTING PRACTICES

2.1 Introduction  
2.2 Review of typical rock bolts and accessories  
2.3 Rock bolting theories  
2.4 Support design philosophy  
2.4.1 Estimation of rock load  
2.4.2 Design of suitable support system  
2.5 Review of rock bolt research
2.6 The concept of constant normal stiffness and its applicability in rock bolting

2.7 Conclusion

Chapter 3. EXPERIMENTAL STUDY OF THE FAILURE MECHANISM OF BOLT/RESIN INTERFACE

3.1 Introduction

3.2 Bolt surface preparation

3.3 Sample casting

3.4 CNS testing apparatus

3.5 Testing of bolt/resin interface

3.6 Shear behaviour of bolt/resin interface

3.6.1 Effect of normal stress on stress paths

3.6.2 Dilation behaviour

3.6.3 Effect of normal stress on peak shear

3.6.4 Effect of cyclic loading on peak shear

3.6.5 Overall shear behaviour of type I and type II bolts

3.6.6 Effect of normal stiffness

3.6.7 Effect of bolt surface profile configuration

3.7 Empirical model for predicting the shear resistance at bolt/resin interface

3.8 Conclusion

Chapter 4. FIELD INVESTIGATION FOR LOAD TRANSFER MECHANISM OF FULLY GROUTED BOLTS

4.1 Introduction

4.2 Site description
4.2.1 West Cliff Colliery (Mine 1) ........................................... 78
4.2.2 Tower Colliery (Mine 2) ............................................... 81

4.3 Instrumentation .............................................................. 86
   4.3.1 Instrumented bolts .................................................. 86
   4.3.2 Sonic probe extensometer ....................................... 90
   4.3.3 Intrinsically safe strain bridge monitor ...................... 92

4.4 Field monitoring and data analysis .................................. 93
   4.4.1 West Cliff Colliery ................................................. 93
   4.4.2 Tower Colliery ........................................................ 100
       4.4.2.1 Load transfer during the panel development and the 101
               longwall retreating phases
       4.4.2.2 Load transfer in the belt and travelling road ........ 103
       4.4.2.3 Behaviour of strata deformation ......................... 107
       4.4.2.4 Impact of strata deformation of the load transfer in the 112
               bolts
       4.4.2.5 Comparison of load transfer in type I and type II bolts 115

4.5 Bolt selection guidelines .............................................. 124
4.6 Conclusions ............................................................... 125

Chapter 5. **SHEAR BEHAVIOUR OF ROCK JOINTS**

5.1 Introduction ............................................................. 127
5.2 Review of unfilled joints .......................................... 128
5.3 Review of unfilled reinforced joints ............................ 133
5.4 Review of infilled joints .......................................... 145
5.5 Concluding remarks .................................................. 153
Chapter 6. SHEAR BEHAVIOUR OF UNFILLED BOLTED JOINTS

6.1 Introduction 154

6.2 Laboratory investigation 154

6.2.1 Selection of model material 154

6.2.2 Preparation of bolted joints 155

6.2.3 CNS direct shear testing apparatus 158

6.2.4 Laboratory experiments 159

6.2.5 Processing of test data 160

6.3 Shear behaviour of non-bolted joints 161

6.3.1 Shear stress 161

6.3.2 Dilation 162

6.3.3 Normal stress 163

6.4 Shear behaviour of bolted joints 164

6.4.1 Shear stress 164

6.4.2 Dilation 165

6.4.3 Normal stress 166

6.5 Strength envelope 168

6.6 Conclusions 169

Chapter 7. SHEAR BEHAVIOUR OF INFILL BOLTED JOINT

7.1 Introduction 171

7.2 Laboratory investigation 171

7.2.1 Sample preparation 171

7.2.2 Testing procedure 174

7.3 Shear behaviour of infilled non-bolted joints 176
7.3.1 Shear stress 176
7.3.2 Normal stress 178
7.3.3 Dilation 179
7.3.4 Stress path 180

7.4 Shear behaviour of infilled bolted joints 182
7.4.1 Shear stress 182
7.4.2 Normal stress 184
7.4.3 Dilation 184
7.4.4 Stress path 186

7.5 Comparison of the shear behaviour of infilled bolted and non-bolted 187
joints
7.5.1 Overall shear behaviour 187
7.5.2 Effect of infill on shear strength 190
7.5.3 Profile of shear plane 192
7.5.4 Strength envelopes 194

7.6 Conclusions 196

Chapter 8. MODELLING OF THE SHEAR BEHAVIOUR OF
REINFORCED JOINTS
8.1 Introduction 198

8.2 Analytical model for shear behaviour of unfilled reinforced joints 199
8.2.1 Definition of Fourier series and its applicability in joint modelling 199
8.2.2 Prediction of variation in normal stress 202
8.2.3 Prediction of shear stress 203

8.3 Analytical model for shear behaviour of infilled reinforced joints 206
8.3.1 Modelling of drop in shear strength 206
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.3.1.1</td>
<td>Impact of infill thickness on peak shear strength</td>
<td>206</td>
</tr>
<tr>
<td>8.3.1.2</td>
<td>Relationship between normalised strength drop and t/a ratio</td>
<td>207</td>
</tr>
<tr>
<td>8.3.1.3</td>
<td>Modelling of normalised strength drop</td>
<td>208</td>
</tr>
<tr>
<td>8.3.2</td>
<td>Modelling of shear strength of infilled bolted joints</td>
<td>211</td>
</tr>
<tr>
<td>8.4</td>
<td>Computer program for calculating Fourier coefficients and the shear strength</td>
<td>212</td>
</tr>
<tr>
<td>8.5</td>
<td>Verification of analytical model</td>
<td>212</td>
</tr>
<tr>
<td>8.6</td>
<td>Numerical model of reinforced joints under CNS conditions</td>
<td>216</td>
</tr>
<tr>
<td>8.6.1</td>
<td>General background</td>
<td>217</td>
</tr>
<tr>
<td>8.6.2</td>
<td>Global reinforcement</td>
<td>217</td>
</tr>
<tr>
<td>8.6.3</td>
<td>Material properties</td>
<td>222</td>
</tr>
<tr>
<td>8.6.4</td>
<td>The continuous yielding model</td>
<td>225</td>
</tr>
<tr>
<td>8.6.5</td>
<td>Conceptual CNS model for reinforced joints</td>
<td>226</td>
</tr>
<tr>
<td>8.6.6</td>
<td>UDEC modelling of reinforced joints under CNS condition</td>
<td>227</td>
</tr>
<tr>
<td>8.6.7</td>
<td>Validation of the model</td>
<td>229</td>
</tr>
<tr>
<td>8.6.8</td>
<td>Comparison of CNL and CNS shear behaviour</td>
<td>234</td>
</tr>
<tr>
<td>8.7</td>
<td>Conclusion</td>
<td>236</td>
</tr>
</tbody>
</table>

**Chapter 9. CONCLUSIONS AND RECOMMENDATIONS**

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.1</td>
<td>Conclusions</td>
<td>237</td>
</tr>
<tr>
<td>9.2</td>
<td>Recommendations for future research</td>
<td>242</td>
</tr>
</tbody>
</table>

**REFERENCES**

**APPENDICES**

Appendix I 

xiii
<table>
<thead>
<tr>
<th>Appendix</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appendix IIA</td>
<td>260</td>
</tr>
<tr>
<td>Appendix IIB</td>
<td>264</td>
</tr>
<tr>
<td>Appendix IIC</td>
<td>265</td>
</tr>
<tr>
<td>Appendix IID</td>
<td>266</td>
</tr>
<tr>
<td>Appendix IIIA</td>
<td>269</td>
</tr>
<tr>
<td>Appendix IIIB</td>
<td>272</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figures</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure 2.1: Suspension theory.</td>
<td>14</td>
</tr>
<tr>
<td>Figure 2.2: Beam building theory.</td>
<td>14</td>
</tr>
<tr>
<td>Figure 2.3: Keying theory.</td>
<td>15</td>
</tr>
<tr>
<td>Figure 2.4: The ground response curve (after Deere et al., 1969).</td>
<td>19</td>
</tr>
<tr>
<td>Figure 2.5: Support management plan (after Strata Control Technology, 1998).</td>
<td>21</td>
</tr>
<tr>
<td>Figure 2.6: Failure modes of rock bolt system a) failure of rock mass, b) failure of bolt/grout/resin interface, and c) failure of bolt (after Littlejohn &amp; Bruce, 1975).</td>
<td>23</td>
</tr>
<tr>
<td>Figure 2.7: Stress distribution along a resin anchor, a) stress situation, and b) theoretical stress distribution (after Farmer, 1975).</td>
<td>25</td>
</tr>
<tr>
<td>Figure 2.8: Load displacement, strain distribution and computed shear stress distribution curves - 500 mm resin anchors in limestone, a) strain distribution, b) stress distribution (after Farmer, 1975).</td>
<td>26</td>
</tr>
<tr>
<td>Figure 2.9: Results from instrumented bolts a) normal stress in the bolt, and b) shear stress at the bolt/grout interface (after Dunham, 1976).</td>
<td>27</td>
</tr>
<tr>
<td>Figure 2.10: Zones of influence of a grouted bolt surrounding a tunnel (after Adali and Russel, 1980).</td>
<td>29</td>
</tr>
<tr>
<td>Figure 2.11: Influence of grouted bolts on tunnel convergence (after Indraratna and Kaiser, 1990).</td>
<td>34</td>
</tr>
<tr>
<td>Figure 2.12: Schematic diagram reflecting the geometry of a rough bolt (after Yazici and Kaiser, 1992).</td>
<td>35</td>
</tr>
<tr>
<td>Figure 2.13: Schematic diagram relating components of bond strength model (after Yazici and Kaiser, 1992).</td>
<td>35</td>
</tr>
<tr>
<td>Figure 2.14: The modified Hoek Cell (after Hyett et al., 1995).</td>
<td>37</td>
</tr>
<tr>
<td>Figure 2.15: Modes of failure of grouted bolt Cell (after Hyett et al., 1995).</td>
<td>38</td>
</tr>
<tr>
<td>Figure 2.16: Detail of short embedment pull test (after Benmokrane et al., 1995).</td>
<td>40</td>
</tr>
<tr>
<td>Figure 2.17: Influence of anchor length on the load-displacement behaviour (after Benmokrane et al., 1995).</td>
<td>41</td>
</tr>
</tbody>
</table>
Figure 2.18: Schematic diagram of bolt anchorage a) long anchor, b) short anchor (after Serrano and Olalla, 1999).

Figure 2.19: Failure mechanism of long and short anchored bolts (after Serrano and Olalla, 1999).

Figure 2.20: Stress components in a small section of a bolt (after Li and Stillborg, 1999).

Figure 2.21: Shear stress along a fully grouted rock bolt subjected to an axial load after decoupling occurs (after Li and Stillborg, 1999).

Figure 2.22: Shear stress along a fully grouted rock bolt subjected to an axial load before decoupling occurs (after Li and Stillborg, 1999).

Figure 2.23: Behaviour of bolted joint in the field under Constant Normal Stiffness

Figure 2.24: Dilation behaviour of joint plane a) two smooth planes, b) bolt and resin interface.

Figure 3.1: Hollow bolt segment.

Figure 3.2: Flattened bolt surface.

Figure 3.3: Bolt surface welded on the top shear box plate.

Figure 3.4: Arrangement of casting resin samples.

Figure 3.5: Cast resin block.

Figure 3.6: Cast resin samples inside the bottom shear box.

Figure 3.7: Line diagram of the CNS apparatus (modified after Indraratna et al., 1997).

Figure 3.8: A closer view of the CNS apparatus (modified after Indraratna et al., 1999).

Figure 3.9: Shear stress profiles of the type I bolt from selected tests.

Figure 3.10: Dilation profiles of the type I bolt from selected tests.

Figure 3.11: Dilation profiles of type I bolt for first cycle of loading.

Figure 3.12: Dilation profiles of type II bolt for first cycle of loading.

Figure 3.13: Shear stress profiles of type I bolt for first cycle of loading.

Figure 3.14: Shear stress profiles of type II bolt for first cycle of loading.

Figure 3.15: Variation of peak shear stress with normal stress for type I bolt.
Figure 3.16: Variation of peak shear stress with normal stress for type II bolt. 67
Figure 3.17: Comparison of stress profile and dilation of type I and type II bolts for first cycle of loading. 68
Figure 3.18: Variation of peak dilation with initial normal stress. 71
Figure 3.19: Comparison of predicted values of shear stresses and the values obtained from laboratory experiments for type I bolt. 74
Figure 3.20: Comparison of predicted values of shear stresses and the values obtained from laboratory experiments for type II bolt. 75

Figure 4.1: Geographical location of West Cliff Colliery (Mine 1), and Tower Colliery (Mine 2). 78
Figure 4.2: Geological sections showing Bulli seam and the associated strata near the instrumentation site at West Cliff Colliery. 79
Figure 4.3: General mine layout of West Cliff Colliery. 80
Figure 4.4: General layout of the panel under investigation indicating instrumentation site at West Cliff Colliery. 80
Figure 4.5: Detail site plan of instrumented bolts at West Cliff Colliery. 81
Figure 4.6: Borehole section showing the geological formation above Bulli seam, Tower Colliery. 82
Figure 4.7: General mine layout of Tower Colliery indicating the panel under investigation. 83
Figure 4.8: Detail layout of the panel under investigation at Tower Colliery. 84
Figure 4.9: Detail layout of the instrumentation site at Tower Colliery. 84
Figure 4.10: Instrumentation naming convention at Tower Colliery, a) for bolts, and b) for extensometers. 85
Figure 4.11: Strain gauge and bolt layout for West Cliff Colliery (Mine 1). 87
Figure 4.12: Strain gauge and bolt layout for Tower Colliery (Mine 2). 88
Figure 4.13: Bolt segment showing channels. 89
Figure 4.14: A section of an instrumented bolt showing the strain gauge and wirings through the silicon gel. 89
Figure 4.15: Various components of an extensometer, a) readout unit, b) probe, and c) photograph showing the process of taking readings in the underground.

Figure 4.16: Photograph showing the process of taking reading with SBM in the underground.

Figure 4.17: Load transferred on the type II bolts, West Cliff Colliery.

Figure 4.18: Load transferred on the bolts installed at the left side of the C/T, West Cliff Colliery.

Figure 4.19: Shear stress developed at the bolt/resin interface of the bolts installed at the left side of the C/T, West Cliff Colliery.

Figure 4.20: Load transferred on bolts installed at the right side of the C/T, West Cliff Colliery.

Figure 4.21: Shear stress developed at the bolt/resin interface of the bolts installed at the right side of the C/T, West Cliff Colliery.

Figure 4.22: Roof condition in the gate roads at Tower Colliery, a) left side, and b) right side.

Figure 4.23: Load transferred on the bolt TRA1, during the panel development and longwall retreating phases, Tower Colliery.

Figure 4.24: Load transferred on the bolt BRA1, during the panel development and longwall retreating phases, Tower Colliery.

Figure 4.25: Maximum load transferred on the bolt BRJ2, for a particular face position, Tower Colliery.

Figure 4.26: Load transferred on the bolts TRJ2 and BRJ2 during the panel development phase, Tower Colliery.

Figure 4.27: Load transferred on the bolts TRJ2 and BRJ2 during the longwall retreating phase, Tower Colliery.

Figure 4.28: Maximum load transferred on the bolts TRJ2 and BRJ2, for a particular face position, Tower Colliery.

Figure 4.29: Strata deformation recorded in the extensometer TR1, Tower Colliery.

Figure 4.30: Strata deformation recorded in the extensometer TR2, Tower Colliery.
List of Figures (contd....)

Figure 4.31: Strata deformation recorded in the extensometer TR3, Tower Colliery.

Figure 4.32: Maximum deformation recorded in the extensometer TR1, for a particular face position, Tower Colliery.

Figure 4.33: Three-dimensional surface generated from the maximum strata deformations recorded in the travelling road, Tower Colliery.

Figure 4.34: Relative displacements between the two consecutive extensometer probes in BR2, Tower Colliery.

Figure 4.35: Load transferred on the bolt BRA2, during the longwall retreating phase, Tower Colliery.

Figure 4.36: Maximum deformation recorded in the extensometer TR2, for a particular face position, Tower Colliery.

Figure 4.37: Maximum load transferred on the bolt TRA2, for a particular face position, Tower Colliery.

Figure 4.38: Load transferred on the type I bolts, installed in the travelling road, Tower Colliery.

Figure 4.39: Load transferred on the type I and type II bolts, installed at the left side of the belt road, Tower Colliery.

Figure 4.40: Shear stress developed at the bolt/resin interface of the type I and type II bolts, installed at the left side of the belt road, Tower Colliery.

Figure 4.41: Load transferred on the type I and type II bolts, installed at the right side of the belt road, Tower Colliery.

Figure 4.42: Shear stress developed at the bolt/resin interface of the type I and type II bolts, installed at the right side of the belt road, Tower Colliery.

Figure 4.43: Load transferred on the type I and type II bolts, installed at the middle of the travelling road, Tower Colliery.

Figure 4.44: Maximum load transferred on type I and type II bolts for a particular face position, installed at the middle of the travelling road, Tower Colliery.
Figure 4.45: Shear stress developed at the bolt/resin interface of the type I and type II bolts, installed at the middle of the travelling road, Tower Colliery.

Figure 4.46: Three-dimensional surface generated from the maximum load transferred on type I and type II bolts, installed in the travelling road, Tower Colliery.

Figure 5.1: Bilinear strength envelope for saw-tooth asperity (after Patton, 1966).

Figure 5.2: Idealised penetration of micro asperities of concrete into rock surface (after Johnston and Lam, 1989).

Figure 5.3: Deformation of bolted joint during shearing (after Indraratna et al., 1999).

Figure 5.4: Shear force versus joint displacement (after Bjurstrom, 1974).

Figure 5.5: Components of shear resistance offered by a bolt (after Bjurstrom, 1974).

Figure 5.6: Arrangement for bolt shear testing (after Hass, 1981).

Figure 5.7: Effect of bolt inclination on the bolt shear resistance (after Aydan and Kwamoto, 1992).

Figure 5.8: Axial and shear stress on bolt at various normal stresses (after Aydan and Kwamoto, 1992).

Figure 5.9: Resistance mechanism of a reinforced rock joint (after Ferrero, 1995).

Figure 5.10: Force components and deformation of a bolt, a) in elastic zone, and b) in plastic zone (after Pellet and Boulon, 1998).

Figure 5.11: Evolution of shear and axial forces in a bolt, a) in elastic zone, and b) in plastic zone (after Pellet and Boulon, 1998).

Figure 5.12: Shear strength of mica infilled joint (after Goodman, 1970).

Figure 5.13: Shear strength of kaolin infilled joint (after Ladanyi and Archambault, 1977).

Figure 5.14: Variation of shear strength with t/a ratio (after Phien-Wej et al., 1990).

Figure 5.15: Effect of t/a ratio on shear strength of infilled joints (after Papalingas et al., 1993).
Figure 5.16: Effect of infill on strength envelope, a) joint with asperity angle 9.5°, and b) joint with asperity angle 18.5° (after Indraratna et al., 1999).

Figure 6.1: Arrangement for sample casting.
Figure 6.2: A typical joint sample.
Figure 6.3: A typical joint sample inside the bottom shear box.
Figure 6.4: Shear stress profile for non-bolted joints.
Figure 6.5: Dilation profile for non-bolted joints.
Figure 6.6: Normal stress profile for non-bolted joints.
Figure 6.7: Shear stress profile for bolted joints.
Figure 6.8: Dilation profile for bolted joints.
Figure 6.9: Normal stress profile for bolted joints.
Figure 6.10: Variation of percentage increase in normal stress with initial normal stress.
Figure 6.11: Strength envelope for bolted and non-bolted joints.
Figure 6.12: Variation of difference between peak shear stresses for bolted and non-bolted joint with initial normal stress.

Figure 7.1: A typical saw tooth joint with infill.
Figure 7.2: Shear strength envelope of clay infill.
Figure 7.3: Process of placing infill on top of joint surface.
Figure 7.4: Installation of bolt perpendicular to the infilled joint.
Figure 7.5: Samples after testing; top: non-bolted, and bottom: bolted.
Figure 7.6: Variation of shear stress for infilled non-bolted joint, a) according to initial normal stress, and b) according to infill thickness.
Figure 7.7: Variation of normal stress for infilled non-bolted joint, a) according to initial normal stress, and b) according to infill thickness.
Figure 7.8: Variation of dilation for infilled non-bolted joint, a) according to initial normal stress, and b) according to infill thickness.
Figure 7.9: Stress paths for infilled non-bolted joint at an initial normal stress of 0.13 MPa.
List of Figures (contd....)

Figure 7.10: Variation of shear stress for infilled bolted joint, a) according to initial normal stress, and b) according to infill thickness. 183
Figure 7.11: Variation of normal stress for infilled bolted joint, a) according to initial normal stress, and b) according to infill thickness. 185
Figure 7.12: Variation of dilation for infilled bolted joint, a) according to initial normal stress, and b) according to infill thickness. 186
Figure 7.13: Stress paths for infilled bolted joint at an initial normal stress of 0.13 MPa. 187
Figure 7.14: Shear stress variation with shear displacement at 2.5 mm infill, for bolted and non-bolted joints. 188
Figure 7.15: Shear strength envelopes of bolted and non-bolted joints at 2.5 mm infill. 189
Figure 7.16: Dilation profiles for bolted and non-bolted joints at 2.5 mm infill. 189
Figure 7.17: Variation of % drop in peak shear strength with infill thickness. 190
Figure 7.18: Variation of shear stress with shear displacement for bolted and non-bolted joints at $\sigma_{no} = 0.63$ MPa. 191
Figure 7.19: Relative location of shear plane through infilled joints at $\sigma_{no} = 1.25$ MPa. 192
Figure 7.20: Stress paths and strength envelopes for 2.5 mm infill thickness ($t/a=0.5$) for bolted and non-bolted joints. 195
Figure 7.21: Stress paths and strength envelopes for 7.5 mm infill thickness ($t/a=1.5$) for bolted and non-bolted joints. 195

Figure 8.1: Periodic function $f(x)$ with a period of $2T$. 200
Figure 8.2: Dilation profile segmented into $m$ equal parts. 201
Figure 8.3: Orientation of bolt with respect to joint plane. 204
Figure 8.4: Variation of peak shear stress with $t/a$ ratio. 207
Figure 8.5: Variation of Normalised Strength Drop (NSD) with $t/a$ ratio. 208
Figure 8.6: Straight line formulation of NSD; a) for non-bolted joint, and b) for bolted joint. 210
Figure 8.7: Comparison of shear stress profiles predicted by the model and from laboratory tests for unfilled joints. 213
Figure 8.8: Comparison of dilation profiles predicted by the model and from laboratory tests for unfilled joints.

Figure 8.9: Comparison of peak shear stress predicted by the model and from laboratory tests for infilled bolted joints.

Figure 8.10: Comparison of normal stress profiles predicted by the model and from laboratory tests for infilled bolted joints (t=1.5 mm).

Figure 8.11: Comparison of dilation profiles predicted by the model and from laboratory tests for infilled bolted joints (t=1.5 mm).

Figure 8.12: Conceptual model representing the mechanical behaviour of fully grouted bolt.

Figure 8.13: Line diagram showing the axial behaviour of bolt in UDEC.

Figure 8.14: A typical bolt element passing through triangular block.

Figure 8.15: Line diagram showing the shear behaviour of grout in UDEC.

Figure 8.16: Conceptual CNS model of reinforced joints.

Figure 8.17: Mesh generation by UDEC.

Figure 8.18: Comparison of shear stress profiles predicted by UDEC model and from laboratory tests.

Figure 8.19: Comparison of dilation profiles predicted by UDEC model and from laboratory tests.

Figure 8.20: Comparison of normal stress profiles predicted by UDEC model and from laboratory tests.

Figure 8.21: Comparison of shear stress profiles predicted by UDEC model under CNL and CNS conditions.
### LIST OF TABLES

<table>
<thead>
<tr>
<th>Tables</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table 2.1: Description of various types of bolt.</td>
<td>11</td>
</tr>
<tr>
<td>Table 2.2: Description of various bolt accessories.</td>
<td>12</td>
</tr>
<tr>
<td>Table 2.3: Geomechanics classification guide for excavation and support in rock tunnel (after Bieniawski, 1987).</td>
<td>18</td>
</tr>
<tr>
<td>Table 3.1: Specification of bolt types.</td>
<td>54</td>
</tr>
<tr>
<td>Table 3.2: Calculated values of a, b and c.</td>
<td>75</td>
</tr>
<tr>
<td>Table 7.1: Dilation or compression of joints for various t/a ratios.</td>
<td>193</td>
</tr>
</tbody>
</table>
LIST OF SYMBOLS AND ABBREVIATIONS

Symbols

- $a$: Asperity height
- $a_n, b_n$: Fourier coefficients
- $A$: Area of joint plane
- $A_b$: Bolt cross sectional area
- $E$: Young's modulus
- $h_{tp}$: Shear displacement at peak shear
- $I$: Asperity angle
- $k_b$: Bolt stiffness
- $k_n$: Bolt-joint composite stiffness
- $k_r$: Joint stiffness
- $k_s$: Joint shear stiffness
- $t$: Infill thickness
- $T$: Period of cycle
- $\alpha, \beta$: Hyperbolic constants
- $\delta_v$: Joint dilation
- $\delta_v(h)$: Joint dilation at any shear displacement
- $\phi_b$: Basic friction angle
- $\sigma_{bn}(h)$: Effective normal stress on the plane of bolt/joint composite at any shear displacement
- $\sigma_c$: Uniaxial compressive strength
- $\sigma_n$: Normal stress
- $\sigma_{no}$: Initial normal stress
- $\sigma_n(h)$: Normal stress at any horizontal displacement
- $\sigma_t$: Tensile strength
- $\tau$: Shear stress
- $\tau_p$: Peak shear stress
\( \tau_{p \text{ infilled}} \)  
Infilled joint shear strength

\( \tau_{p \text{ unfilled}} \)  
Unfilled joint shear strength

\( \Delta \tau_p \)  
Drop in peak shear strength

\( \theta \)  
Inclination of bolt with respect to the joint plane

**Abbreviations**

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>BSM</td>
<td>Bond Strength Model</td>
</tr>
<tr>
<td>CNL</td>
<td>Constant Normal Load</td>
</tr>
<tr>
<td>CNS</td>
<td>Constant Normal Stiffness</td>
</tr>
<tr>
<td>JCS</td>
<td>Joint wall Compressive Strength</td>
</tr>
<tr>
<td>JRC</td>
<td>Joint Roughness Coefficient</td>
</tr>
<tr>
<td>NATM</td>
<td>New Austrian Tunnelling Method</td>
</tr>
<tr>
<td>NSD</td>
<td>Normalised Strength Drop</td>
</tr>
<tr>
<td>OBE</td>
<td>Optimum Beaming Effect</td>
</tr>
<tr>
<td>RRU</td>
<td>Reinforced Rock Unit</td>
</tr>
<tr>
<td>SBM</td>
<td>Strain Bridge Monitor</td>
</tr>
<tr>
<td>UDEC</td>
<td>Universal Distinct Element Code</td>
</tr>
<tr>
<td>UOW</td>
<td>University of Wollongong</td>
</tr>
<tr>
<td>USBM</td>
<td>United States Bureau of Mines</td>
</tr>
</tbody>
</table>
Chapter 1

INTRODUCTION

1.1 GENERAL BACKGROUND

Since early civilisation, man has been involved in the development of stable infrastructure, and in the process, a number of methods have been developed and evolved to stabilise the man-built structures. In mining engineering and underground rock structures, bolting is considered to be one such technique for improving the structural stability. According to Kareby (1995), rock bolting in various forms has been used as early as the nineteenth century. Development has progressed from wooden dowels (nineteenth century), to expansion shell bolts (early twentieth century), and subsequently to cement and resin grouted rebars (1950) and friction stabilisers such as swellex and split set bolts (1980).

However, in Australia, rock bolts were not used extensively until the construction of Snowy Mountain Hydroelectric Scheme in 1948. Since then, stabilisation of rock mass by systematic bolting has become a de facto standard of stabilising the superficial workings (e.g. slopes, dams etc.) as well as the underground excavations. Hundreds of millions of bolts are installed annually in various construction projects,
and a recent survey revealed the worldwide usage in excess of 500,000,000 units per annum (Windsor, 1997).

In recent years, the range of application of rock bolts has been broadened both in mining and civil engineering. This is due to advances made in the understanding of bolt failure mechanisms as well as the improvement attained in strata control technology. This is clearly demonstrated by the increased use of rock reinforcement in underground excavations as an alternative to traditional forms of support such as timber, crossbar, wooden chock etc.

The rock bolt research gained momentum following the introduction of New Austrian Tunnelling Method (NATM) in the early sixties, where fully grouted bolts were employed as a major support component in rock tunnels. Bjurström (1974) and Hass (1981) conducted systematic laboratory studies on the behaviour of fully grouted bolts to predict the shear strength of bolted joints as a function of the friction angle, normal stress and bolt inclination. It was concluded that inclined bolts were stiffer, and that they contributed more effectively towards enhanced shear strength of discontinuities than the bolts installed perpendicular to the joint plane. Based on shear testing of model gypsum joints, Dight (1982) found that normal stress acting on the joint plane had little influence on the bolt shear resistance, and that joint dilatancy was related to the bolt inclination. Indraratna and Kaiser (1990) presented a comprehensive analytical model for predicting the stability of circular tunnels with grouted bolts, where the convergence was determined as a function of the bolt spacing, the tunnel diameter and the bolt friction. In a recent paper, Pellet and Egger
Chapter 1: Introduction

(1996) introduced an analytical model for evaluating the role of bolts on the shear strength of rock joints. The interaction of the axial and shear forces mobilised in the bolt as well as the large plastic displacements of the bolt during the loading process were considered in the model. However, none of these studies include the influence of bolt surface (profile) configuration on the load transfer mechanism of grouted bolts.

During the last three decades, various researchers have extensively examined in the laboratory, the shear strength parameters of both natural and artificial infilled rock joints (e.g. Goodman, 1970; Kanji, 1974; Ladanyi & Archambult, 1977; Lama, 1978; Barla et al., 1985; Bertacchi et al., 1986; Pereira, 1990; Phien-wej et al., 1990 and de Toledo & de Freitas, 1993). All these tests were carried out under the conventional Constant Normal Load (CNL) conditions, and not under Constant Normal Stiffness (CNS) conditions, as there was insufficient understanding of the relevance of CNS testing in underground mining conditions.

Johnstone & Lam (1989), Skinas et al. (1990) and Indraratna et al. (1999) have described the importance of CNS condition to simulate the actual shear behaviour in the field. For non-planar discontinuities, shearing often results in dilation as one asperity rides over another. If the surrounding rock mass is unable to deform sufficiently, then an inevitable increase in the normal stress occurs during shearing. Therefore, the CNL condition is unrealistic in circumstances where the normal stress in the field changes considerably during the shearing process, such as in some underground mining situations. The deformation behaviour of a bolted joint is
governed by the applied normal stress, the stiffness of the joint-bolt composite, and the frictional properties of the joint interface. The CNS method is more realistic than the conventional CNL approach in such situations as discussed by Indraratna et al. (1999).

Leichnitz (1985) was the first to report on the laboratory studies of rock joints under CNS condition, and he verified that both the shear force and dilation were functions of the normal force and shear displacement. Since then, similar findings have been reported by Johnstone & Lam (1989), Van Sint Jan (1990), Ohnishi and Dharmaratne (1990), Skinas et al. (1990) and Haberfield & Johnstone (1994). More recently, Indraratna et al. (1999) discussed the shear behaviour of soft joints under CNS condition for both clean and infilled joints, where an analytical model was developed to predict the shear behaviour of soft joints using Fourier transforms.

Despite the significant advances made in strata reinforcement technology, the mechanisms of bolt-rock interaction are still complex and they remain to be a grey area for research. To the best of writer's knowledge, no research work has been reported on the influence of the surface geometry of bolts on the load transfer mechanism, and the effect of infill material on the overall shear strength of bolted joints under CNS condition. Accordingly, the research study reported in this thesis is aimed to address various issues in order to provide solutions to the complex problem of bolt and rock interaction. In particular, the study will focus on the shear strength of bolt/resin/rock interface and the shear behaviour of infilled bolted joints.
1.2 KEY OBJECTIVES

The objectives of the present research work include:

- The study of the shear behaviour and load transfer mechanism in rock/grout/bolt interfaces both in the laboratory and in the field.
- The study of the influence of bolt surface geometry on the load transfer mechanism of fully grouted bolts.
- The study of the shear behaviour of both unfilled and infilled bolted joint mainly under CNS condition.
- Development of an analytical model to predict the shear behaviour of both unfilled and infilled bolted joint, and

1.3 OUTLINE OF THE THESIS

The thesis consists of nine chapters and is organised as follows:

This chapter has provided a brief introduction of the development and evolution of rock bolting technology. Chapter 2 is concerned with the present support design philosophy for rock engineering. It contains an examination of the current roof
bolting practices in the field and the popular rock bolting theories currently employed in practice. A brief description of available approaches for the determination of rock loads and the associated design procedures are also discussed.

Chapter 3 describes the experimental studies undertaken on the shear behaviour and failure mechanism of the bolt/resin interface of fully grouted bolts. It contains the principles and the detailed design of the large-scale CNS direct shear apparatus, sample preparation and the experimental procedures adopted for the study. The shear behaviour of flattened bolt surface of two most popular bolt types is examined under CNS condition with initial normal stress levels of 0.1 to 7.5 MPa. The influence of bolt profile configuration on the effective shear strength at the bolt/resin interface is described together with the failure mechanism of the bolt/resin interface.

Chapter 4 includes the analysis of field investigation carried out in two coal mines in the Illawarra region of NSW, Australia. As extensive instrumentation procedure of bolts, site description, field installation procedures, description of field monitoring instruments and data collection are contained in this chapter. The results from eighteen instrumented bolts installed at three different mine sites are elucidated. The chapter also contains the guidelines for selecting a suitable bolt surface profile under various confining stress conditions.

Chapter 5 is devoted to reviewing the influencing factors governing the shear behaviour of both unfilled and infilled bolted joints. It contains a review of selected past studies, the existing models simulating the shear behaviour of joints and limited
literature on infilled joints tested under CNS condition. To the best of writer's knowledge, no suitable literature was available to report on the shear behaviour of infilled bolted joint under CNS condition.

Chapter 6 presents the result and discussion of the shear behaviour of both unfilled bolted and non-bolted joints under CNS condition. The interpretation of data relates to stress paths, stress-strain relationship, strength envelope and dilation behaviour of unfilled joints. The bolt contribution on the shear strength of unfilled joint is also described.

Chapter 7 presents the results and discussions pertinent to the CNS behaviour of both infilled bolted and non-bolted joints. The analysis of the test results was conducted in relation to the shear strength, dilation and normal stress responses. The effect of infill thickness on the overall shear strength of both bolted and non-bolted joint is explained. The influence of bolt in relation to the infill thickness on the shear strength of bolted joint is also described. The concept of Normalised Strength Drop (NSD) associated with the increase in infill thickness for bolted and non-bolted joints is introduced.

Chapter 8 introduces an analytical model representing the general shear behaviour of bolted joints applicable to both unfilled and infilled bolted joints under CNS condition. Based on the Fourier transforms, the dilation response during shearing process of bolted joint is accurately modelled, and finally, a model for the prediction of shear behaviour of unfilled bolted joint is developed. The shear behaviour of
infilled bolted joint is modelled using the unfilled joint model and the NSD. A comparative study of the current model predictions with laboratory results, as well as the results from UDEC analysis are presented at the end of the chapter.

Conclusions and recommendations are given in the final Chapter 9. It summarises the findings of this research study in relation to; (a) the influence of bolt surface geometry on the overall shear behaviour of bolt/resin interface, (b) the selection of a suitable bolt profile configuration depending on the prevailing in-situ stress condition, (c) the shear behaviour of unfilled bolted joints, and (d) the shear behaviour of infilled bolted joints. Recommendations on the possible future lines of investigation are also presented in this chapter. A list of references and appendices follow Chapter 9.
Chapter 2

ROCK BOLTING PRACTICES

2.1 INTRODUCTION

Since its introduction as a support system to mining and civil engineering, rock bolting has evolved through different phases of technological transformations. In the early stages, wooden dowels were used to prevent roof falls in coal faces in the British Coal Mines (Beyl, 1945-46). In recent years, a variety of rock bolting systems have been developed and, in fact, they have become the most common method of ground support in most Australian mines and most rock engineering operations.

The early studies on the fundamental of bolting techniques date back to the studies carried out by USBM in 1948. Since then, substantial research has been carried out, covering various aspects of bolting technology such as design philosophy, bolt configuration, bolt load transfer mechanism, installation procedures for effective ground stabilisation and so on.

This chapter is concerned with the literature review of rock bolting technology. It provides a critical review of the bolt types, its application and theories related to
bolting and contemporary research work carried out in this line. Particular emphasis is given to the behaviour of rock bolt under Constant Normal Stiffness (CNS) condition, as CNS is the primary basis of the research work undertaken in this thesis.

2.2 REVIEW OF TYPICAL ROCK BOLTS AND ACCESSORIES

Although a number of different types of rock bolts and its variants are in use today, they may be broadly classified into the following four major groups:

- Mechanically anchored rock bolts
- Frictional rock bolts
- Grouted rock bolts
- Grouted cable bolts

Table 2.1 shows a brief description of each type of bolt together with the relative advantages and disadvantages.

In addition to the bolts used in a reinforcement system, a number of other components are also used to make reinforcement systems more effective, as well as for obtaining better load transfer characteristics. These include face plate, anti-friction washer, w-strap, wire mesh etc. Table 2.2 shows a brief description of each component.
Table 2.1: Description of various types of bolt.

<table>
<thead>
<tr>
<th>Bolt type</th>
<th>Applicability</th>
<th>Capacity (tons)</th>
<th>Advantages</th>
<th>Disadvantages</th>
<th>Diagram</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mechanical</td>
<td>Moderately hard to hard rock</td>
<td>10 - 16</td>
<td>• Inexpensive • Immediate support possible • High capacity in hard rocks.</td>
<td>• Limited to use in hard rock condition • Long-term stability is affected by slippage.</td>
<td>SHELL LENGTH WEDGE LENGTH</td>
<td>Used to be most common type of bolt during early stages.</td>
</tr>
<tr>
<td>Friction</td>
<td>Medium to hard rock</td>
<td>10 - 14</td>
<td>• Simple installation • Can accommodate large displacement • Reusable.</td>
<td>• Relatively expensive • Hole diameter is critical for installation • Not very corrosion resistive.</td>
<td></td>
<td>Split set bolt is the most recent development (1980s).</td>
</tr>
<tr>
<td>Grouted bolt</td>
<td>Universal</td>
<td>15 - 25</td>
<td>• High corrosion resistance • Durable • Consistent.</td>
<td>• Installation is time critical • Costlier than mechanical bolt.</td>
<td></td>
<td>Presently most common type of bolt.</td>
</tr>
<tr>
<td>Grouted cable</td>
<td>Universal</td>
<td>up to 50</td>
<td>• Same as grouted bolt • Can be installed in long sizes up to 15 m • Riffling effect.</td>
<td>• Tensioning of cable requires a special installation procedure.</td>
<td></td>
<td>Versatile in nature.</td>
</tr>
</tbody>
</table>
Table 2.2: Description of various bolt accessories.

<table>
<thead>
<tr>
<th>Accessories</th>
<th>Usage</th>
<th>Diagram</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Face plate</td>
<td>To uniformly distribute the load at bolt collar.</td>
<td></td>
<td>Used with all types of bolt. Dome shape is the most common.</td>
</tr>
<tr>
<td>Anti-friction washer</td>
<td>To reduce the friction between the nut and face plate.</td>
<td></td>
<td>Increases the torque-tension ratio. Common for all bolt types.</td>
</tr>
<tr>
<td>W-strap</td>
<td>Pulled into rock surface by the bolt to conform major irregularities.</td>
<td></td>
<td>Provide a large surface confinement to any loose rock between bolts.</td>
</tr>
<tr>
<td>Wire mesh</td>
<td>To prevent injury to personnel and damage to equipment from small pieces of rock or spalled flakes.</td>
<td></td>
<td>Used right up to the face to avoid any accident at intervals between 1 to 1.5 m.</td>
</tr>
</tbody>
</table>
2.3 ROCK BOLTING THEORIES

Many researchers have put forward their opinions as to how roof bolts act to support the immediate roof. The most widely accepted theories include:

- Suspension theory
- Beam building theory
- Keying theory

Whenever an underground opening is made, the immediate strata directly overhead tend to sag. If not properly and adequately supported, the laminated immediate roof could separate from the main roof and may collapse. Roof bolts, in such situations, nail the immediate roof to the self-supporting main roof by the tension applied to the bolts. Suspension theory assumes that the rock near the excavation is weak and fractured, however, rocks that are high above in the roof are relatively stronger in comparison with the immediate roof rocks. The length of the bolt should be long enough to penetrate into the stable rock above the immediate roof. Bolts should be strong enough to hold the dead weight of fractured rock beneath the stable strata. Figure 2.1 shows the action of bolts as the basis of the suspension theory.

In many cases, the strong, self-supporting main roof may not necessarily be located within the distance that ordinary roof bolts can reach to form firm anchors for suspension. In such a situation, it is assumed that the bolt acts by laminating thin and weak individual layers of rock together into a thicker and stronger beam of rock.
Such a system will transmit horizontal shear force from one layer to another and reduce the horizontal displacement between layers. When the theory is applied in respect to tensioned bolts, the frictional resistance between the thin layers is increased due to the increased normal force acting perpendicular to the bedding plane. Non-tensioned fully grouted bolts will also transmit shear forces between the bedding planes as a result of the shear stiffness of the bolt/grout composite. Figure 2.2 explains the action of bolts according to the beam building theory.

Figure 2.1: Suspension theory.

Figure 2.2: Beam building theory.
When the roof strata are highly fractured and blocky, or the immediate roof contains one or several set of joints with different orientations, roof bolting significantly increases frictional forces along fractures, cracks and weak planes. Sliding and/or separation along weak interface is thus prevented or reduced as shown in Figure 2.3. The keying effect mainly depends on active bolt tension or under favourable circumstances, passive tension due to rock mass movement. It has been observed that bolt tension produces stress in the stratified roof, which are compressive both in the direction of the bolt axis and orthogonal to the bolt. Superposition of the compressive area around each bolt forms a continuous compressive zone in which tensile stresses are offset and the shear strength is improved.

Figure 2.3: Keying theory.
2.4 SUPPORT DESIGN PHILOSOPHY

The primary concern regarding the support design is that whether the structure under consideration is self-supporting, and if not, what kind of strategies must be followed for the overall stability of the structure during the entire life span of the roof. The selection of support members is not only closely associated with their mechanical functions but also with the availability, suitability and economic viability of the support elements. Currently, the support design philosophy consists of two fundamental steps:

- Estimation of rock load
- Design of suitable support system

2.4.1 Estimation of Rock Load

There are several approaches to determine the load on rock and these are classified into following groups:

- Rock mass classification approach
- Semi-empirical approach
- Structural defect approach
- Rock-support interaction approach

In the rock mass classification approach, the immediate roof is classified into different categories depending on the values assigned to various structural and
geological parameters of the roof rock and the method of excavation (Terzaghi, 1946; Deere et al., 1969; Bieniawski, 1973; Barton et al., 1973). Over the years, the rock mass classification approach has gained increasing popularity because of its simplicity of use. Table 2.3 show the guidelines for geomechanics classification for excavation and support in rock tunnels as proposed by Bieniawaski, 1987.

According to the *semi-empirical approach* it is assumed that a ground arch around the opening occurs and the boundaries of the loosening zone are restricted between the ground arch and the excavation boundary. The fundamental reasoning for this assumption originates from the fact that the rock mass is incapable of resisting tensile stresses and is sufficiently strong while under compression. The original theory was proposed by Janssen (1895) for soils and was applied to tunnelling by Terzaghi (1946).

The *structural defect approach* involves the determination of the dimensions of potentially unstable blocks or layers of rock in relation to the special orientation of discontinuities and the geometry of the opening. The total potential volume of rock prone to fall will vary depending on the number of sets of joint planes and the tensile strength of the rock or the shear strength of joint planes, whichever is weaker. When the number of joint sets is two or more, the determination of potentially unstable blocks sliding or toppling into the excavation, can be determined by block theory (Wittke, 1964; John, 1970; Londe and Vormeringer, R., 1970, Goodman and Shi, 1984). This theory can effectively evaluate the possibility of sliding or falling of unstable rocks and resulting loads.
Table 2.3: Geomechanics classification guide for excavation and support in rock tunnel (after Bieniawski, 1987).

<table>
<thead>
<tr>
<th>Rock mass class</th>
<th>Excavation</th>
<th>Support</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Rock bolt</td>
</tr>
<tr>
<td>I. Very good rock RMR: 81-100</td>
<td>Full face:</td>
<td>Generally no support required</td>
</tr>
<tr>
<td></td>
<td>- 3 m advance</td>
<td></td>
</tr>
<tr>
<td>II. Good rock RMR: 61-80</td>
<td>Full face:</td>
<td>Locally bolts in crown</td>
</tr>
<tr>
<td></td>
<td>- 1.0-1.5 m advance;</td>
<td>- 3 m long, spaces 2.5 m with occasional wire mesh</td>
</tr>
<tr>
<td></td>
<td>- Complete support 20 m from face</td>
<td></td>
</tr>
<tr>
<td>III. Fair rock RMR: 41-60</td>
<td>Top heading and bench:</td>
<td>Systematic bolts 4 m long</td>
</tr>
<tr>
<td></td>
<td>- 1.5-3 m advance in top heading;</td>
<td>- Spaced 1.5-2 m in crown and walls with wire mesh in crown</td>
</tr>
<tr>
<td></td>
<td>- Commence support after each blast;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Complete support 10 m from face</td>
<td></td>
</tr>
<tr>
<td>IV. Poor rock RMR: 21-40</td>
<td>Top heading and bench:</td>
<td>Systematic bolts 4-5 m long</td>
</tr>
<tr>
<td></td>
<td>- 1.0-1.5 m advance in top heading;</td>
<td>- Spaced 1-1.5 m in crown and with wire mesh</td>
</tr>
<tr>
<td></td>
<td>- Install support concurrently with excavation - 10 m from face</td>
<td></td>
</tr>
<tr>
<td>V. Very poor rock RMR: &lt;20</td>
<td>Multiple drifts:</td>
<td>Systematic bolts 5-6 m</td>
</tr>
<tr>
<td></td>
<td>- 0.5-1.5 m advance in top heading;</td>
<td>- Long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert</td>
</tr>
<tr>
<td></td>
<td>- Install support concurrently with excavation;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Shotcrete as soon as possible after blasting</td>
<td></td>
</tr>
</tbody>
</table>
In the *rock support interaction approach*, the rock load is based on the interaction between rock mass and support system. The basic idea of this approach was first suggested by Fenner (1938) and was introduced to the real tunnelling practice by Rabcewicz (1964) and was modified by Deere et al. (1969). Figure 2.4 shows a typical ground reaction curve for rock tunnels. The ground reaction curve displays the load that must be applied to the roof or the walls of the tunnel to prevent further movement. This approach later gave birth to a new technique of tunnelling known as the *New Austrian Tunnelling Method (NATM)*. The principle of this approach is well illustrated by a ground response - support reaction curve. The load supported by the support members will be the one at which ground response curve intersects the support reaction curve.

![Diagram](image)

**Figure 2.4: The ground response curve (after Deere et al., 1969).**
2.4.2 Design of Suitable Support System

Once the rock load has been calculated using any of the methods discussed above, a suitable support system may be designed depending on the availability, working conditions and the economic viability of the support elements. It has been a common practice to support the longwall gate roads by means of full column resin bolts along with wire mesh and W-straips. The support density depends on the estimated rock load and the length and capacity of the bolt. Figure 2.5 shows the flow chart of a typical support management plan for a mine. It consists of four distinct phases:

- Design
- Implementation
- Monitoring
- Response

The design phase includes the selection of a suitable support system depending on the existing geotechnical data at hand. In the implementation phase all the designed support elements are put forward into the appropriate locations. Once the support system is installed, the next step is to monitor the performance of the support system. The response of the support design team will depend upon the analysis of data obtained from monitoring. The necessary action required at various stages is explained in Figure 2.5.
Figure 2.5: Support management plan (after Strata Control Technology, 1998).
2.5 REVIEW OF ROCK BOLT RESEARCH

Traditionally, geomechanical problems are collectively examined using a combination of experimental, analytical and numerical methods. However, the selection of any appropriate combination of approaches is dependent on the prevailing properties of the resulting physical phenomenon. Numerous investigations have been carried out in the past to study the effect of reinforcement by rock bolts. Some of the selected research work, mainly in the area of load transfer and failure of rock bolt system, are described in the following paragraphs. The research work in relation to the shear strength of reinforced joint is reviewed in Chapter 5.

The first systematic study on the failure of rock bolt system was carried out by Littlejohn and Bruce (1975). Three modes of failure of rock bolt system has been suggested, which are attributed to:

- Failure of rock mass,
- Failure of rock bolt, and
- Failure of bolt/grout/rock interface.

All three modes of failure are shown in Figure 2.6. The estimated bolt load was given by:

\[ P = 0.1 \sigma_c \pi r L \]  

(2.1)
where,

\[ P = \text{maximum normal force in bolt}, \]

\[ \sigma_c = \text{uniaxial compressive strength of surrounding rock}, \]

\[ r = \text{borehole radius}, \] and

\[ L = \text{bond length}. \]

Figure 2.6: Failure modes of rock bolt system a) failure of rock mass, b) failure of bolt/grout/resin interface, and c) failure of bolt (after Littlejohn & Bruce, 1975).

Farmer’s (1975) mathematical model on stress distribution along the encapsulated bolt length compared favourably with pullout test results of the instrumented bolt. Farmer (1975) proposed that, the shear stress in resin annulus is a function of bolt extension and modulus of rigidity of the grout as follows:

\[ \tau_s = \frac{\xi_s}{(R - a)} G_g ; \text{When} \quad R - a < a \quad (2.2) \]
\[ \tau_x = \frac{\xi_x}{a \ln (R/a)} G_g \] ; When \( R > a > a \) \hspace{1cm} (2.3)

where,

\( \tau_x = \) shear stress in the resin annulus,
\( \xi_x = \) extension in the bolt,
\( a = \) radius of bolt,
\( x = \) distance along the length of bolt starting at free end of grout,
\( R = \) radius of borehole, and
\( G_g = \) shear modulus of grout.

Shear stress distribution along a typical resin anchor was given by the following equation and is shown in Figure 2.7:

\[ \frac{\tau}{\sigma_0} = 0.1 e^{-\left(\frac{0.2x}{a}\right)} \] \hspace{1cm} (2.4)

where,

\( \sigma_0 = \) stress on bolt at \( x \) equal to zero.

The pull test showed that there were stress variations along the bolt length. Maximum stress values were recorded at the free end and the minimum at the far end. This was attributed to the interaction between the bolt and grout interface. The
theoretical stress distributions were compared favourably with the laboratory results as shown in Figure 2.8.

Figure 2.7: Stress distribution along a resin anchor, a) stress situation, and b) theoretical stress distribution (after Farmer, 1975).
Figure 2.8: Load displacement, strain distribution and computed shear stress distribution curves - 500 mm resin anchors in limestone, a) strain distribution, b) stress distribution (after Farmer, 1975).

Farmer's mathematical model was supported by the work of Dunham (1976) when carrying out pullout tests on strain gauged instrumented bolts, particularly at small pulling force. However, as the load increased, the contact between the bolt and the
grout was lost resulting into a drop in the value of shear stress along the increasing length (see Figure 2.9).

![Figure 2.9: Results from instrumented bolts a) normal stress in the bolt, and b) shear stress at the bolt/grout interface (after Dunham, 1976).](image)

**Use of Plasticity and Limit Equilibrium Theories**

Adali and Russel (1980) conducted studies on physical models of circular tunnel with grouted bolts to calculate the size and number of bolts. Outer stress conditions were
used as an input to the model. A potential function $V$ was introduced in the basic equilibrium equation, and accordingly:

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} + \Omega = 0$$  \hspace{1cm} (2.5)$$

$$\sigma_r = \frac{1}{r} \frac{d\phi}{dr} + \nu, \quad \sigma_\theta = \frac{d^2\phi}{dr^2} + \nu \quad \text{and} \quad \frac{dv}{dr} = -\Omega$$

where,

$\sigma_r$ = radial stress,

$\sigma_\theta$ = tangential stress,

$r$ = distance from centre to point of interest,

$\Omega$ = radial body force per unit volume, not due to gravity,

$V$ = potential function, and

$\phi$ = stress function.

The whole space was divided into three regions as shown in Figure 2.10. These regions were described as the fractured zone or the protective zone, influence zone of grouted bolts, and the outer zone respectively. A solution was provided for the state of stress and deformation in each region. Although an elastic model is a rather rough approximation of the actual situation and tend to predict values that are higher than the actual values, nevertheless, it gave a general idea on the properties of a tunnel boundary conditions supported by grouting. It was found that if the rock is not highly
fractured, the protective zone becomes larger and the pressure on the tunnel wall increases, thus requiring greater number of bolts for effective support.

Lang and Bischoff (1981) studied the stability of reinforced rock structure and developed the concept of Reinforced Rock Unit (RRU) which was analogous to the vissour in the old masonry structure. Using limit analysis procedure and characteristic properties for the rock and rock reinforcement, functional equations were developed for the stability of reinforced rock units relative to one another. Along with the solution for the design and utilisation of rock reinforcement system, the following equation was formulated to determine the stability index for the resulting reinforced rock structure:

Figure 2.10: Zones of influence of a grouted bolt surrounding a tunnel (after Adali and Russel, 1980).
Chapter 2: Rock Bolting Practices

\[ T = \alpha \frac{\gamma AR}{K\mu} \left( 1 - \frac{c}{\gamma R} \right) \left\{ \frac{h\mu}{\gamma R} \left[ 1 - e^{-K\mu D/R} \right] \right\} \]  \hspace{1cm} (2.6)

where,

- \( T \) = bolt tension,
- \( \alpha \) = factor depending on the time of bolt installation after excavation (0.5 - 1.0),
- \( \gamma \) = unit weight of rock,
- \( R \) = shear radius = \( A/P \),
- \( A \) = area of reinforced rock unit supported by a bolt,
- \( P \) = shear perimeter of reinforced rock unit,
- \( K \) = ratio of average horizontal to average vertical stress,
- \( \mu = \tan(\varphi) \),
- \( \varphi \) = angle of internal friction,
- \( c \) = apparent cohesion of the rock mass,
- \( h \) = in-situ horizontal stress,
- \( D \) = height of de-stressed rock above the surface of the opening, and
- \( l \) = length of bolts.

It was shown that adequate and safe design of a rock reinforcement system requires consideration of the following;

- rock properties,
• in situ stress field,
• bolt material (steel) properties,
• bolt pattern: length and spacing,
• bolt tension,
• time of installation,
• modes of failure, and
• mining procedures and operations.

Tao and Chen (1983) proposed a simple mathematical model for predicting stress distribution along the bolt based on field studies on the stress distribution along instrumented bolt in yielding rock. The concept of neutral point was subsequently introduced. In yielding ground a fully grouted bolt is essentially divided into a pick-up length and an anchor length. The pick-up length restrains the rock from deforming, whereas the anchor length is restrained by rock. The shear strain distribution ($\tau_z$) is given by:

$$\tau_z = \frac{1}{\pi d} \frac{dQ(z)}{dz}$$  \hspace{1cm} (2.7)

where,

\[ d = \text{bolt diameter, and} \]

\[ Q(z) = \text{axial stress distribution along the bolt.} \]

The neutral point of the bolt is given by the limit equilibrium analysis:
\[ \rho = \frac{L}{\ln[1 + (L/a)]} \]  

where,

\[ \rho = \text{radial distance to the neutral point}, \]
\[ L = \text{bolt length}, \text{ and} \]
\[ a = \text{tunnel radius}. \]

Indraratna and Kaiser (1990) studied the failure mechanism of surrounding rock mass in a tunnel reinforced by fully grouted bolts. The basic equilibrium equation for tunnel was modelled by introducing equivalent strength parameters in relation to bolt density. The following model was proposed along with a complete set of solutions for stress distribution, strain and deformation surrounding the tunnel:

\[ \frac{d\sigma_r}{dr} + \frac{(1-m^*)\sigma_r}{r} = \frac{\sigma_{cr}^*}{r} \]  

where,

\[ \sigma_r = \text{radial stress field}, \]
\[ r = \text{distance from tunnel centre to point of interest}, \]
\[ m^* = m(1+\beta), \]
\[ m = \text{internal friction parameter used in Mohr-Coulomb failure criteria}, \]
\[ \beta = \text{bolt density parameter and is given by limit equilibrium}: \]
\[
\frac{\pi d \lambda a}{S_L . S_T}
\]

\(d\) = bolt diameter,

\(\lambda\) = friction factor for bolt grout interaction,

\(a\) = tunnel radius,

\(S_L\) = longitudinal bolt spacing,

\(S_T\) = circumferential bolt spacing,

\(\sigma_{cr^*}\) = \(\sigma_{cr} (1 + \beta)\), and

\(\sigma_{cr}\) = equivalent post peak compressive strength, upon yielding.

The above analytical model was verified by conducting studies on laboratory simulated physical models of a tunnel supported with fully grouted bolts. It was shown that the greater the bolt density, the lesser was the tunnel wall convergence (see Figure 2.11).

Yazici and Kaiser (1992) conducted studies on bond strength of grouted cable bolts and presented a conceptual model for fully grouted cable bolts, known as Bond Strength Model (BSM). The simplified bolt surface was assumed to be zigzag in shape as shown in Figure 2.12, which does not necessarily represent the actual bolt surface profile. The concept of bond strength model is explained in Figure 2.13, which consists of four quadrants. The first quadrant shows the variation of bond strength with axial displacement. The second quadrant relates to the pressure at the bolt-grout interface to the bond strength. The third quadrant shows the relation
between axial and lateral displacement, and lastly the fourth quadrant depicts the behaviour of grout during dilation.

Figure 2.11: Influence of grouted bolts on tunnel convergence (after Indraratna and Kaiser, 1990).
Figure 2.12: Schematic diagram reflecting the geometry of a rough bolt (after Yazici and Kaiser, 1992).

Figure 2.13: Schematic diagram relating components of bond strength model (after Yazici and Kaiser, 1992).
Using the equality of small angle tangents to the angles in radia, the bond strength was proposed in terms of friction and dilation angle given by:

\[
\tau = \sigma \cdot \tan \left\{ i_o \left[ 1 - \left( \frac{\sigma}{\sigma_{\text{lim}}} \right)^{\beta} \right] + \phi \right\}
\]  

(2.10)

where,

- \( \tau \) = bond strength,
- \( \sigma \) = radial stress at the bolt grout interface,
- \( \sigma_{\text{lim}} \) = limiting radial stress,
- \( \beta \) = coefficient of reduction of dilation angle, and
- \( \phi \) = friction angle between steel and grout.

The ultimate bond strength at the bolt grout interface was limited by the grout strength. It was concluded that shearing off of the rough edges in the resin/rock annulus eventually prevents further dilation, but the pressure on the bolt remains constant during the pulling of the bolt. The major shortcoming of the above theory is that it assumes the bolt surface is zigzag in nature (see Figure 2.12), which is not realistic.

Hyett et al. (1995) proposed a constitutive law for bond failure of fully grouted cable bolts based on a series of pull tests performed in a modified Hoek cell as shown in Figure 2.14. A fully grouted seven-wire strand cable was used and the confining pressure at the outside cement annulus was maintained constant. The bond strength was shown to increase with the confining pressure. The radial dilation was attributed
to an unscrewing failure mechanism along the majority of the test section. Test results were used to develop a frictional-dilational model for cable bolt failure. Three failure mechanisms were proposed; dilational slip, non-dilational unscrewing and shear failure of the cement flutes as shown in Figure 2.15. The axial forces corresponding to the three failure mechanisms were by a series of equations as follows.

1. 15.2 mm seven-wire strand
2. Type 10 Portland cement annulus
3. Pressure vessel end cap
4. Specimen end cap
5. 15 mm PVC tube for de-bonding
6. ABS pipe to support end of specimen and overcome end-effects
7. Neoprene bladder
8. Cantilever strain gauge arms
9. High pressure electrical feed through
10. High pressure fitting
11. Pressure transducer

Figure 2.14: The modified Hoek Cell (after Hyett et al., 1995).
Rotational slip (unscrewing)

- For dilational slip, after splitting of the cement annulus,

\[ F_a = A_1 P_1 \tan (\phi_{gs} + i) \]  \hspace{1cm} (2.11)

- For non-dilational unscrewing, and

\[ F_a = \frac{A_1 P_1 \tan \phi_{gs}}{\sin \alpha} + Q \]  \hspace{1cm} (2.12)
• For shear failure of the cement flutes,

$$F_a = A_i \left( \tau_0 + P_1 \tan \phi_g \right)$$  \hspace{1cm} (2.13)$$

where,

$F_a$ = axial load on cable,
$A_i$ = apparent cable grout interface contact area,
$P_1$ = radial pressure at $r = r_1$, inner radius of cement annulus,
$\phi_{gs}$ = friction angle between grout and steel,
$i$ = dilation angle of the cable grout interface,
$\alpha$ = pitch angle,
$Q$ = component of axial force related to untwisting of the cable,
$\tau_0$ = Mohr-Coulomb cohesion intercept for the grout, and
$\phi_g$ = internal angle of friction for the grout.

The bond strength was found to depend on the grout quality; the radial stiffness of the borehole wall and the mining induced distressing.

Benmokrane et al. (1995) conducted laboratory studies on pull test of short encapsulation length (see Figure 2.16). Six types of cement based grouts and two types of rock anchors were used. The following empirical equation was derived for the estimation of anchor pullout resistance for the given embedment length:
\[ P = a + b \frac{AL}{\phi} \]  

(2.14)

where,

\( P \) = ultimate pull-out load,

\( AL \) = anchored length,

\( \phi \) = diameter of anchor, and

\( a, b \) = constants, depend on grout and anchor type.

It was noted that the slip at the unloaded end started at approximately 80% of the failure load (see Figure 2.17). Based on the pullout results, an analytical model for the bond-stress-slip relation was proposed for both the threaded bars and the stranded cables respectively:
\[ \tau = mS + n \]  

(2.15)

where,

\[ \tau \] = shear bond stress at anchor grout interface,

\[ S \] = slip between anchor and grout, and

\[ m, n \] = coefficients which depend on the type of anchor, grout and stages of shear.

Figure 2.17: Influence of anchor length on the load-displacement behaviour (after Benmokrane et al., 1995).
Stankus and Peng (1996) proposed a new concept for roof support design using Optimum Beaming Effect (OBE). The OBE was defined as the roof beam that has no separation above or within the bolted range and having the shortest possible bolt. The OBE concept was considered to be an effective tool with regard to saving in material cost, material handling and drilling time in addition to improving the roof control problems.

Serrano and Olalla (1999) described the tensile resistance of rock anchors based on Euler’s variational method and Hoek and Brown (1980) rock mass failure criterion. Depending on the slenderness ratio ($L/D$; $L$ = anchor length, $D$ = anchor diameter), two types of failure surfaces were obtained e.g. short and long (see Figure 2.18). Figure 2.19 shows the deformation behaviour of short and long anchors respectively. In case of long anchors, nearly all the failure surface coincided with a cylindrical surface defined by its diameter and length. In the case of short anchors, convex surfaces were obtained. For both short and long anchors, a formula was proposed for the pullout strength. The values of ultimate pullout strength, depending on the rock type and its RMR indices, were obtained and compared with published data. Because of its dependency on RMR, the model is empirical in nature, and thus would be site specific.

Li and Stillborg (1999) proposed an analytical model for predicting the behaviour of bolts under three different conditions; (a) for bolts subjected to a concentrated pull load in pullout test, (b) for bolts installed in uniformly deformed rock masses, and (c) for bolts subjected to the opening of individual rock joints. The stress component in
a small section of a bolt is shown in Figure 2.20 and the conceptual model for force equilibrium equation along the axis of the bolt was expressed by:

\[
\tau_b = -\frac{A}{\pi d_b} \frac{d\sigma}{dx}
\]  

(2.16)

where,

\(\tau_b\) = shear stress at the bolt interface,
\(A\) = cross sectional area of the bolt,
\(d_b\) = bolt diameter,
\(\sigma\) = axial stress on the bolt, and
\(x\) = small length of bolt considered.

Figure 2.18: Schematic diagram of bolt anchorage a) long anchor, b) short anchor (after Serrano and Olalla, 1999).
Figure 2.19: Failure mechanism of long and short anchored bolts (after Serrano and Olalla, 1999).
The model was based on the description of the mechanical coupling at the interface between the bolt and the grout medium for grouted bolts, or between the bolt and the rock for frictionally coupled bolts. It was suggested that for rock bolts in pull out tests, the shear stress of the interface attenuates exponentially with increasing distance from the point of loading when the deformation is compatible across the interface. Decoupling was found to start at the loading point when the applied load was large enough, and then propagated towards the far end of the bolt with a further increase in the applied load. Accordingly, the shear stress distribution along a fully grouted rock bolt before and after coupling is shown in Figures 2.21 and 2.22 respectively. The magnitude of the shear stress on the decoupled bolt section was dependent on the coupling mechanism at the interface. For fully grouted bolts, the shear stress on the decoupled section was lower than the peak shear strength of the interface, while for frictionally coupled bolts, it was approximately the same as the
peak shear strength. It was also revealed that the face plate plays a significant role in enhancing the reinforcing effect.

Figure 2.21: Shear stress along a fully grouted rock bolt subjected to an axial load after decoupling occurs (after Li and Stillborg, 1999).

Figure 2.22: Shear stress along a fully grouted rock bolt subjected to an axial load before decoupling occurs (after Li and Stillborg, 1999).

Tadolini et al. (2000) studied the performance of high capacity tensioned cables for supporting a tailgate. A combination of tensioned cable systems consisting of 4.5 m 7 strand cables (18 mm diameter) with a designed capacity of 58 tons was found to successfully maintain the roof of longwall tailgate. The formation of roof guttering was largely reduced by the application of tensioned cables across the failed zone. It
was also found that the application of tension on the point-anchored cable supports moved the roof upward during jacking process, but failed to make the roof stiffer.

Witthaus and Ruppel (2000) described the successful implementation of bolting in very deep and highly stressed multi-seam mining conditions. Different planning elements such as the results of the observation of physical models, numerical models (using FLAC and GEDRU) for calculation of stress-distribution, laboratory and underground support testing were used. The knowledge of in-situ strata behaviour including geotechnical classifications was used to successfully implement the support system. More emphasis was given to regular monitoring of support performance and re-estimation of various parameters using deviations of actual parameters from the planned ones. In the same context, Wittenberg and Ruppel (2000) emphasised the importance of a comprehensive quality management system for successful application of rock bolts in highly stress conditions.

2.6 THE CONCEPT OF CONSTANT NORMAL STIFFNESS AND ITS APPLICABILITY IN ROCK BOLTING

In the past, various research work have been carried out on reinforced rock joints in the laboratory using conventional direct shear apparatus, where the normal stress acting on the joint interface was considered to be constant throughout the shearing process. If the shearing surface is smooth enough and offering little or negligible dilation, then the testing under the conventional Constant Normal Load (CNL) condition is adequate to represent shear behaviour. For non-planar discontinuities,
however, shearing often results in dilation as one asperity rides over another. If the surrounding rock mass is unable to deform sufficiently, then an inevitable increase in the normal stress occurs during shearing which is dependent on the applied initial normal stress, the stiffness of the host medium and the dilation of the shearing interface. Therefore, the CNL condition is unrealistic in circumstances where the normal stress in the field changes considerably during the shearing process, for example, as in the case in many underground mining situations. Thus, shearing of rough surface no longer takes place under CNL condition; rather it is the stiffness of the surrounding rock mass that remains relatively constant and, governs the shear behaviour. Accordingly, this mode of shearing is defined as shearing under Constant Normal Stiffness (CNS).

Johnstone & Lam (1989), Skinas et al. (1990) and Indraratna et al. (1999) have described the importance of CNS condition to simulate the actual shear behaviour in the field. Figure 2.23 shows a schematic diagram of a bolted joint, whose deformation behaviour is governed by the applied normal stress, the stiffness of the joint-bolt composite, and the frictional properties of the joint interface. The CNS method is more realistic than the conventional CNL approach in such situations. Figure 2.24 describes the shearing process of bolt/resin/rock interface. As indicated earlier, in case of smooth surface (see Figure 2.24a) no dilation occurs and hence, both CNL and CNS will produce same result. However, in case of shearing of bolt/resin/rock interface, the rough bolt surface rides over the resin and/or rock (see Figure 2.24b), hence, the CNS is more appropriate than the CNL. Indraratna et al.
(1999), and Indraratna & Haque (2000) have discussed many other situations where CNS is more realistic as compared to CNL.

\[
\begin{align*}
\sigma_n &= f(k_n, \delta_v) \\
k_n &= f(k_r, k_b)
\end{align*}
\]

\(k_r \text{ and } k_b\) = rock and bolt stiffness

\(\delta_v\) = joint dilation

Figure 2.23: Behaviour of bolted joint in the field under Constant Normal Stiffness.

\[\sigma_n = \sigma_{no} \quad \text{a) CNL/CNS} \]

- Smooth surface
- No dilation

\[\sigma_n = \sigma_{no} + k.\delta_v/A \quad \text{b) CNS} \]

- Rough surface
- Dilation

Figure 2.24: Dilation behaviour of joint plane a) two smooth planes, b) bolt and resin interface.
2.7 CONCLUSION

A brief description of the existing rock bolting practices has been given. Today, fully grouted rock bolts have become the industry standard for supporting underground roadways. The successful implementation of roof support requires a detailed plan indicating geotechnical behaviour of host medium, estimated rock load, support design, monitoring of support performance and re-establishment of whole cycle, if necessary.

To date, most of the rock bolt research is concerned mainly with the estimation of rock load and the load transfer mechanisms via the bolt-ground interaction. Some of the reported research work aimed at studying the failure mechanism of the bolt/resin interface or the rock mass itself. Surprisingly, the study on the influence of bolt surface geometry on the failure mechanism of bolt/grout interface is scarce, and the limited research work in this line was reported by Yazici and Kaiser (1992) with an assumption of a zigzag bolt surface, which is uncommon in mines. Accordingly, the need to effectively investigate the impact of bolt surface geometry on the failure mechanism of bolt/resin/rock interface is of paramount importance. Thus, this thesis is aimed at examining the failure mechanism of such bolt/resin interface under Constant Normal Stiffness condition, which in underground mining situations is considered to be more realistic as compared to the conventional Constant Normal Load condition.
Chapter 3

EXPERIMENTAL STUDY OF THE FAILURE MECHANISM OF BOLT/RESIN INTERFACE

3.1 INTRODUCTION

One of the most important parameters in rock/resin/bolt load transfer mechanism is the surface condition of the bolt, i.e. the bolt surface roughness. The study of load transfer mechanism would remain incomplete without the relationship between the bolt surface roughness and the shear behaviour of bolt/resin interface, as the surface roughness dictates the level of interlocking between the bolt and the resin surfaces. A realistic outcome of such study was considered possible if the tests were conducted under Constant Normal Stiffness (CNS) condition, because the normal stress acting on the bolt/resin interface changes, however the stiffness remains constant during shearing process. The existing CNS testing machine as described by Indraratna et al. (1997) was modified to incorporate the surface profile of the bolt for direct shear testing of cast resin blocks.

A series of tests were conducted on the flattened bolt surface of two most popular bolt types currently being used in Australian coal mines, and are referred to in this
thesis as type I and type II bolts, respectively. The flattened bolt surface was used to print the image of the bolt surface on cast resin samples, such that the resin and bolt surfaces exactly match with each other during testing. The testing program included the shearing of bolt/resin interface of two bolt types at different initial normal stress levels, ranging between 0.1 and 7.5 MPa, at a constant shearing rate of 0.5 mm/min. Each sample was subjected to several cycles of loading to observe the effect of repeated loading on the bolt/resin interface.

This chapter contains the detailed design of a large-scale CNS apparatus for direct shear testing, the preparation of the bolt surface and the resin samples used for testing. A brief review of the laboratory testing program is discussed together with the relevant instrumentation and monitoring procedures implemented. Also discussed is the analysis of test data and the development of an empirical model to predict the shear strength of bolt/resin interface.

3.2 BOLT SURFACE PREPARATION

A 100 mm length of a bolt was selected for the surface preparation for CNS shear testing. The specified length of bolt was cut and then drilled through as shown in Figure 3.1. The hollow bolt segment was then cut along the bolt axis from one side and preheated to open up into a flat surface as shown in Figure 3.2. The surface features of the bolt (ribs) were carefully protected while opening up the bolt surface. The flattened surface of the bolt was then welded on the bottom plate of the top shear
box of the CNS testing machine as shown in Figure 3.3. Although these flattened bolt surfaces may not ideally represent the complex behaviour of circular shaped bolt surface observed in the field, nevertheless, they still provided a simplified basis for evaluating the impact of the bolt surface geometry on the shear resistance offered by a bolt. Table 3.1 shows the detailed specification of two types of bolts used for the study.

Figure 3.1: Hollow bolt segment.

Figure 3.2: Flattened bolt surface.
Figure 3.3: Bolt surface welded on the top shear box plate.

Table 3.1: Specification of bolt types.

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Core Diameter (mm)</th>
<th>Finished Diameter (mm)</th>
<th>Rib Spacing (mm)</th>
<th>Rib Height (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I</td>
<td>21.7</td>
<td>24.4</td>
<td>28.5</td>
<td>1.35</td>
</tr>
<tr>
<td>Type II</td>
<td>21.7</td>
<td>23.2</td>
<td>12.5</td>
<td>0.75</td>
</tr>
</tbody>
</table>

3.3 SAMPLE CASTING

The welded bolt surface on the bottom plate of the top shear box was used to print the image of bolt surface on cast resin samples. The arrangement for casting the sample is shown in Figure 3.4. For obvious economic reasons, the samples were cast in two parts. Nearly three-fourth of the mould was cast with high strength casting plaster and the remaining one-fourth was topped up with a chemical resin commonly
used for bolt installation in coal mines. A curing time of two weeks was allowed for all specimens before testing was carried out. The properties of the hardened resin after two weeks were: uniaxial compressive strength ($\sigma_c$) = 76.5 MPa, tensile strength ($\sigma_t$) = 13.5 MPa, and Young’s modulus ($E$) = 11.7 GPa. The cured plaster showed a consistent $\sigma_c$ of about 20 MPa, $\sigma_t$ of about 6 MPa, and $E$ of 7.3 GPa. Such model materials were suitable to simulate the behaviour of a number of jointed or soft rocks, such as coal, friable limestone, clay shale and mudstone, and were based on the ratios of $\sigma_c/\sigma_t$ and $\sigma_c/E$ applied in similitude analysis (Indraratna, 1990).

Figure 3.5 shows a typical cast sample and Figure 3.6 shows such a sample placed in the shear box of the CNS testing machine. The resin samples prepared in this way matched exactly with the bolt surface, allowing a close representation of the bolt/resin interface in practice.
3.4 CNS TESTING APPARATUS

Figure 3.7 is a general view of the CNS testing apparatus used for the study. Designed and built in house, the equipment was basically a modified version of the
type used by researchers earlier at Monash University (Johnstone and Lam, 1989).

The modified CNS apparatus incorporated the bolt surface profile in the top shear box.

![CNS shear apparatus diagram](image)

**Figure 3.7:** Line diagram of the CNS apparatus (modified after Indraratna et al., 1997).

As can be seen in Figure 3.7 the equipment consisted of a set of two large shear boxes to hold the samples in position during testing. The size of the bottom shear box
is 250x75x100 mm while the top shear box is 250x75x150 mm. A set of four springs are used to simulate the normal stiffness ($k_n$) of the surrounding rock mass. The top box can only move in the vertical direction along which the spring stiffness is constant (8.5 kN/mm). The bottom box is fixed to a rigid base through bearings, and it can move only in the shear (horizontal) direction. The desired initial normal stress ($\sigma_{no}$) is applied by a hydraulic jack, where the applied load is measured by a calibrated load cell. The shear load is applied via a transverse hydraulic jack, which is connected to a strain-controlled unit. The applied shear load can be recorded via strain meters fitted to a load cell. The rate of horizontal displacement can be varied between 0.35 and 1.70 mm/min using an attached gear mechanism. The dilation and the shear displacement of the joint are recorded by two LVDT’s, one mounted on top of the top shear box and the other is attached to the side of the bottom shear box. Figure 3.8 shows a closer view of the CNS apparatus.

Figure 3.8: A closer view of the CNS apparatus (modified after Indraratna et al., 1997).
3.5 TESTING OF BOLT/RESIN INTERFACE

A total of 12 samples were tested for two different types of bolt surface at initial normal stress ($\sigma_{no}$) levels ranging from 0.1 to 7.5 MPa. Each sample for bolt type I was subjected to five cycles of loading in order to observe the effect of repeated loading on the bolt/resin interface. Samples for bolt type II were subjected to only three cycles of loading, as it was found that the stress profile did not vary significantly after the third cycle of loading. The stress profile, as described above, is defined as the variation of shear (or normal) stress with shear displacement for various cycles of loading. The $\sigma_{no}$ applied to the samples represented typical confining pressures, which might be expected in the field, especially in shallow mining conditions. A constant normal stiffness of 8.5 kN/mm (or 1.2 GPa/m when applied to a flattened bolt surface of 100 mm length) was applied via an assembly of four springs mounted on top of the top shear box. The simulated stiffness was found to be representative of the soft coal measure rocks. An appropriate strain rate of 0.5 mm/min was maintained for all shear tests. A sufficient gap (less than 10 mm) was allowed between the upper and lower boxes to enable unconstrained shearing of the bolt/resin interface.

3.6 SHEAR BEHAVIOUR OF BOLT/RESIN INTERFACE

3.6.1 Effect of Normal Stress on Stress Paths

Figure 3.9 shows the shear stress profiles of the bolt/resin interface for selected normal stress conditions for the type I bolts. The difference between stress profiles
for various loading cycles was negligible at low values of $\sigma_{no}$ (Figure 3.9a). This was gradually increased with increasing value of $\sigma_{no}$ reaching a maximum between 3 and 4.5 MPa (Figure 3.9b). Beyond a 4.5 MPa confining pressure, the difference between stress profiles for the loading cycles I and II decreased again (Figure 3.9c). A similar trend was also observed for the type II bolt surface (not shown in figure).

At low $\sigma_{no}$ values, the relative movement between the bolt/resin surfaces caused an insignificant shearing and slickensiding of the resin surface, thus keeping the surface roughness nearly unchanged. For each additional cycle of loading, the shear stresses marginally decreased, especially in the peak shear stress region (see Figure 3.9a). However, as the value of $\sigma_{no}$ was increased, the shearing of the resin surface was also increased, and the difference in stress profiles for various cycles of loading became significant. At moderate normal stress levels (between 3 and 4.5 MPa), the shearing and slickensiding of the contact surface was not sufficient to smoothen the resin surface excessively in the first cycle, and further cyclic loading was required to cause a significant difference in the observed stress profiles. This is indicated by the continuously decreasing gap between the shear stress profiles in Figure 3.9b. As the initial normal stress was increased beyond this level, the difference in shear stress profiles for various cycles of loading was decreased due to excessive shearing and smoothening of the resin surface, after the first cycle of loading. At this stage, no further decrease in stress profiles was observed, i.e. after the second cycle of loading as shown in Figure 3.9c. The difference in stress profile after the third cycle was negligible at almost all normal stress levels, thus indicating no change in surface properties after the third cycle of loading.
Figure 3.9: Shear stress profiles of the type I bolt from selected tests.
3.6.2 Dilation Behaviour

Figure 3.10 shows the dilation profiles from selected tests for the bolt type I. The dilation of the bolt/resin interface for both bolt types was characterised by the initial contraction, and then subsequent increase in dilation for all the loading cycles. This initial contraction may be due to the early degradation of very fine irregularities of the resin surface. The dilation was reduced with increasing value of $\sigma_{no}$ (e.g., peak dilations in Figure 3.10b are smaller as compared to those in Figure 3.10a), and correspondingly, the increase in normal stress from its initial value was also found to decrease with increasing $\sigma_{no}$, as per Equation 3.1 described later. This gives an indication that the behaviour of rock/resin interface at CNS condition may approach the CNL condition at very high $\sigma_{no}$ values, where no significant increase in normal stress is expected to be observed from its initial value throughout the shearing process. As expected, the gap between the dilation profiles for various loading cycles remained constant at low initial normal stress levels (Figure 3.10a), and progressively decreased at moderate normal stress levels (Figure 3.10b).

For the first cycle of loading, Figures 3.11 and 3.12 show the variation of dilation with shear displacement at various normal stresses for type I and type II bolts, respectively. For various values of $\sigma_{no}$, the maximum dilation occurred at a shear displacement of 17 - 18 mm and 7 - 8 mm, for type I and type II bolts, respectively (Figures 3.11 and 3.12). The distance between the ribs for both bolt types is shown in Table 3.1. Therefore, it may be concluded that the maximum dilation occurred at a shear displacement of about 60% of the bolt rib spacing.
Figure 3.10: Dilation profiles of the type I bolt from selected tests.

Figure 3.11: First loading cycle dilation profiles for the type I bolt at various $\sigma_{\text{no}}$ values.
3.6.3 Effect of Normal Stress on Peak Shear

At various normal stresses, Figures 3.13 and 3.14 show the variation of shear stress with shear displacement for the first cycle of loading, for both type I and type II bolts, respectively. The shear displacement for peak shear stresses increased with increasing value of $\sigma_{no}$ for both bolt types. This was due to the increased amount of resin surface shearing with the increasing value of $\sigma_{no}$. However, there was a gradual reduction in the difference between the peak shear stress profiles with increasing value of $\sigma_{no}$. The shear displacement required to reach the peak shear strength is a function of the applied normal stress and the surface properties of the resin, assuming that the geometry of the bolt surface remains constant for a particular type of bolt as evident from Figures 3.13 and 3.14.
3.6.4 Effect of Cyclic Loading on Peak Shear

For various loading cycles, Figures 3.15 and 3.16 show the variation of peak shear stress with normal stress applied for type I and type II bolts, respectively. For the type
Chapter 3: Experimental Study of the Failure Mechanism of Bolt/Resin Interface

I bolt surface, the graphs of cycle I through cycle III show a bi-linear trend, whereas the graphs representing cycles IV and V show only a linear trend. For the type II bolt surface, only cycle I shows a bi-linear trend and cycles II and III show a linear trend. At low initial normal stress, the shearing of resin surface is negligible, and hence, the rate of increase of peak shear stress with respect to normal stress is high. At higher normal stress, the degree of shearing of resin surface is greater, and some of the energy is thus utilised to shear off the resin surfaces. As a result, there is retarded rate of increase in peak shear stress with respect to the normal stress. As the samples are loaded repeatedly, the resin surfaces become smoothened reducing the surface roughness, and as a result, the rate of increase of peak shear stress is likely to remain constant with respect to the normal stress.

![Graph](image)

Figure 3.15: Variation of peak shear stress with normal stress for type I bolt.
3.6.5 Overall Shear Behaviour of Type I and Type II Bolts

Figure 3.17 shows the stress profiles of both type I and type II bolts plotted together for the first cycle of loading. The following observations were noted:

- The shear stress profiles around peak were similar for both bolt types. However, slightly higher stress values were recorded for the bolt type I at low normal stress levels, whereas slightly higher stress values were observed for the bolt type II at high normal stress levels, in most cases.

- Post peak shear stress values are higher for the bolt type I indicating better performance in the post-peak region.

- Shear displacements at peak shear are higher for the bolt type I indicating the safe allowance of more roof convergence before instability stage is reached.

- Dilation is greater in the case of bolt type I.
Chapter 3: Experimental Study of the Failure Mechanism of Bolt/Resin Interface

Shear Stress Profile of Type I and Type II Bolts

Normal Stress Profile of Type I and Type II Bolts

Dilation Profile of Type I and Type II Bolts

Figure 3.17: Comparison of stress profile and dilation of type I and type II bolts for first cycle of loading.
3.6.6 Effect of Normal Stiffness

The laboratory experiments were carried out with spring assembly with an effective stiffness of 8.5 kN/mm. In practice, the stiffness of resin/rock system will be usually higher than the laboratory simulated stiffness. As the stiffness increases, the effective normal stress on the bolt/resin interface at any point of time will also increase, as per the following equation:

\[
\sigma_n = \sigma_{no} + \frac{k_n \delta_v}{A}
\]  

(3.1)

where,

\[
\begin{align*}
\sigma_n &= \text{effective normal stress}, \\
\sigma_{no} &= \text{initial normal stress}, \\
k_n &= \text{system stiffness}, \\
\delta_v &= \text{dilation}, \text{ and} \\
A &= \text{area of the bolt surface}.
\end{align*}
\]

In general, higher values of effective normal stresses should be observed for the type I bolt as compared to the type II bolt as long as the confining pressure remains low.

Higher values of effective normal stress will have a direct positive impact on the peak shear stress values and, therefore, when installed in the field, the type I bolt would outperform the type II bolt, particularly at low confining pressure conditions.
However, in deep mining conditions, where the high stress conditions prevail, the reverse situation should be observed. This phenomenon is explained in Section 3.6.7.

3.6.7 Effect of Bolt Surface Profile Configuration

In a relative displacement situation, however minute the movement is between resin and bolt surface, the complete bolt surface is no longer in contact with the whole resin surface. The contact zone between the resin and the bolt surface at that point is usually restricted to the bolt rib and its surrounding area only. At low normal stress condition, the stress concentration on the resin surface around the bolt ribs is not sufficient to cause resin failure. However, at high normal stress levels, the stress concentration on the resin surface due to the projected ribs of the bolt surface reaches a value which is greater than the compressive strength of the resin, causing it to crush around that zone.

The stress concentration is more evenly distributed on the resin surface for the bolt type II with rib spacing of 12.5 mm, as compared to the bolt type I with a rib spacing of 28.5. At low normal stress values, the stress concentration on the resin surface is not high enough to crush it, thus its integrity remains intact. As a result, the bolt with deeper ribs will offer higher resistance due to higher dilation (see Figure 3.18). However, at a high normal stress level where the concentrated stress around ribs is high enough to crush the resin surface, the bolt that has lower rib spacing provides a more evenly distributed stress on the resin surface. This is indicated by a relatively flatter diminishing peak dilation curve for type II bolts as compared to type I bolts.
(Figure 3.18). In other words, the bolts with lower rib spacing would offer a greater resistance at high normal stress conditions.

![Graph showing variation of peak dilation with initial normal stress.](image)

Figure 3.18: Variation of peak dilation with initial normal stress.

### 3.7 Empirical Model for Predicting the Shear Resistance at Bolt/Resin Interface

Based on the above study a mathematical model was developed to predict the shear resistance of bolt/resin interface at any normal stress within the range of experimental stresses. The shear stress at any particular point of time could be expressed as a function of the shear displacement, the normal stress and the surface properties of the bolt. Examination of the shapes of shear stress profiles at various normal stresses suggests the following empirical relationship (Daniel and Wood, 1980):
\[ \tau = a \cdot x^b \cdot e^{c \cdot x} \]  \hspace{1cm} (3.2)

where,

\[ \tau = \text{predicted shear stress}, \]
\[ x = \text{shear displacement}, \]
\[ e = \text{natural log base}, \]
\[ a = f (\text{normal stress}), \]
\[ b = f (\text{normal stress}), \] which takes into account the effect of shearing of asperities at high normal stress values, and
\[ c = f (\text{surface properties}) \] i.e. constant for a particular set of bolt-resin interface and is equal to -0.092 and -0.29 for both type I and type II bolt, respectively.

Assuming that the parameters \( a \) and \( b \) are constant for any particular normal stress, the peak shear stress can be obtained by differentiating Equation 3.2 with respect to the shear displacement and equalling it to zero. Therefore, for \( \tau \) to be maximum,

\[ \frac{d\tau}{dx} = 0 \quad \text{or} \quad x = -\frac{b}{c} \]  \hspace{1cm} (3.3)

By incorporating the value of \( x \) in Equation 3.2,

\[ \tau_{\text{max}} = a \cdot \left( -\frac{b}{c \cdot e} \right)^b \]  \hspace{1cm} (3.4)
For the set of data generated from the laboratory experiments the following relationship was obtained based on best-fit regression analysis.

If $N =$ initial normal stress:

*for type I bolt;*

\[
\begin{align*}
  a &= 0.425 + 2.79 \, e^{-1.9 / N} \\ 
  b &= 0.405 + 0.08 \, N \quad \text{for } N \leq 3 \, \text{MPa} \\
  &= 0.580 + 0.0265 \, N \quad \text{for } N > 3 \, \text{MPa}
\end{align*}
\]  

*for type II bolt;*

\[
\begin{align*}
  a &= 0.45 + 3.48 \, e^{-1.29 / N} \\ 
  b &= 0.525 + 0.142 \, N \quad \text{for } N \leq 4.5 \, \text{MPa} \\
  &= 0.95 + 0.0467 \, N \quad \text{for } N > 4.5 \, \text{MPa}
\end{align*}
\]

The two different functional values of $b$ could be explained from Figures 3.15 and 3.16 where the relationship is bi-linear. The first and steeper segment is continued up to 3 MPa for type I bolt and 4.5 MPa for type II bolt, after which the second and flatter segment represents the effect of shearing of asperities at higher normal stress levels. The above anomaly necessitates the use of different functional values of $b$ for
different normal stress levels. Table 3.2 shows the calculated values of a, b and c using the above equations for various normal stress levels.

Figures 3.19 and 3.20 show the predicted shear stresses for any shear displacement at any particular normal stress, and the corresponding values based on laboratory observations for both type I and type II bolts. The predicted values are in close agreement with the experimental values throughout the loading period.

Figure 3.19: Comparison of predicted values of shear stresses and the values obtained from laboratory experiments for type I bolt.
Figure 3.20: Comparison of predicted values of shear stresses and the values obtained from laboratory experiments for type II bolt.

Table 3.2: Calculated values of a, b and c.

<table>
<thead>
<tr>
<th>Normal stress (MPa)</th>
<th>Type I bolt</th>
<th>Type II bolt</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>0.1</td>
<td>0.43</td>
<td>0.41</td>
</tr>
<tr>
<td>1.5</td>
<td>1.2</td>
<td>0.52</td>
</tr>
<tr>
<td>3.0</td>
<td>1.9</td>
<td>0.65</td>
</tr>
<tr>
<td>4.5</td>
<td>2.25</td>
<td>0.699</td>
</tr>
<tr>
<td>6.0</td>
<td>2.46</td>
<td>0.739</td>
</tr>
<tr>
<td>7.5</td>
<td>2.59</td>
<td>0.78</td>
</tr>
</tbody>
</table>
3.8 CONCLUSION

It can be inferred from this experimental study that:

- The shear behaviour of the bolt surface at various confining pressures directly affects the load transfer mechanism from the rock to the bolt.
- The type I bolt offered higher shear resistance at low confining pressure (below 6.0 MPa), whereas, the type II bolt offered greater shear resistance at high normal stress conditions exceeding 6.0 MPa. This was attributed to the surface profile configuration of the bolt i.e., the spacing and the depth of the rib.
- The bolt with deeper rib offered higher shear resistance at low normal stress conditions, while the bolt with closer rib spacing offered higher shear resistance at high normal stress conditions.
- The impact of repeated loading on the effective shear resistance of the bolt/resin interface was influenced by the magnitude of the applied normal stress, the number of loading cycles and the surface geometry of the bolt.
- The maximum dilation occurred at a shear displacement of nearly 60% of the rib spacing.
- The bolt type I showed better performance than that of bolt type II when considering the shear behaviour at low normal stress, dilational aspects, and the post-peak behaviour.
- The empirical model could predict the complete shear stress profile at various normal stress conditions, which was in close agreement with the laboratory results.
4.1 INTRODUCTION

In an attempt to verify the laboratory findings, with respect to the influence of bolt surface profiles on load transfer mechanisms under different testing environments, a program of field investigations were undertaken in two local coal mines (West Cliff and Tower Collieries) in the Southern Coalfields of Sydney Basin, NSW, Australia. Both selected sites were at the development headings, serving the newly developed retreating longwall faces. The examination of the bolt behaviour was undertaken with respect to the prevailing in-situ stress conditions, both in magnitudes and the stress directions, affecting the stability of the development headings. Two different types of fully instrumented bolts were installed, and the methodology of their installation, positioning, and regular monitoring is the subject of detailed discussion in this chapter. The chapter also contains the guidelines for selecting a suitable bolt surface profile under various confining stress conditions.
4.2 SITE DESCRIPTION

4.2.1 West Cliff Colliery (Mine 1)

West Cliff Colliery is located in the Southern Coalfields of Sydney Basin, NSW, Australia (see Figure 4.1). The mine was selected as the first site for field investigation. West Cliff Colliery currently extracts coal from the Bulli seam at a depth of 465m and produces around 1.45 million tonnes per annum from the 200m wide longwall face and development operations. The average seam thickness is 2.7m, and a representative geological section close to the experimental site is shown in Figure 4.2. The roof strata consisted of sandstone/shale/siltstone combination, which could be considered to be stable.

Figure 4.1: Geographical location of West Cliff Colliery (Mine 1), and Tower Colliery (Mine 2).
Figure 4.2: Geological sections showing Bulli seam and the associated strata near the instrumentation site at West Cliff Colliery.

Figure 4.3 shows the general layout of the mine and the location of the longwall panel. Detailed layout of the panel under investigation (panel 512) is shown in Figure 4.4. Six, 2.1m long instrumented bolts (three of each type I and type II bolts as used in the laboratory experiments) were installed in the longwall panel cut-through (C/T 7). Figure 4.5 shows the details of the instrumented bolt installation pattern at C/T 7. In order to avoid excessive and non-uniform loading of the bolts (because of wide excavation at the cut-through intersections), the bolts were installed near the middle length of the cut-through. The regular pattern of bolting in the site was 6 bolts in a
row and the spacing between the rows was 1m. At the test site, three bolts were
installed in each row, and in all there were two rows of instrumented bolts, with each
row containing one type of bolts. Also shown in Figure 4.5 are the magnitude and the
direction of the prevailing principal horizontal stress. Following installation of
instrumented bolts in the month of January 1999, field data was subsequently
collected at regular intervals, until the longwall face past the instrumented cut-
through in June 1999.

Figure 4.3: General mine layout of West Cliff Colliery.

Figure 4.4: General layout of the panel under investigation indicating instrumentation
site at West Cliff Colliery.
4.2.2 Tower Colliery (Mine 2)

Tower Colliery is also located in the Illawarra Coal Measures of NSW, Australia, and the geographical location for the mine is shown in Figure 4.1. The mine was selected for the second site of field monitoring because of its proximity to the University of Wollongong. The average seam thickness of the Bulli seam in the instrumented site was 3.3m at a depth of 515 m, and the average annual production from the 230 m wide longwall face and other development operations is around 1.4 million tonnes. Figure 4.6 shows the geological section of a borehole close to the experimental site, illustrating the immediate roof stratification above the Bulli seam. The roof rock consisted of mainly sandstones, mudstones, siltstones and occasional weak bands of shale, and is classified as moderate to strong roof. Figure 4.7 shows the general
layout of the mine and the location of longwall panel. Twelve instrumented bolts, each 2.4 m long were installed in two adjacent main entry roads to longwall panel (panel 18) as shown in Figure 4.8. The general arrangement for instrumented bolt installations in both gateroads was the same as at West Cliff Colliery, and is shown in Figure 4.9.

Figure 4.6: Borehole section showing the geological formation above Bulli seam, Tower Colliery.
Figure 4.7: General mine layout of Tower Colliery indicating the panel under investigation.
Chapter 4: Field Investigations

Figure 4.8: Detail layout of the panel under investigation at Tower Colliery.

Figure 4.9: Detail layout of the instrumentation site at Tower Colliery.
In addition to the instrumented bolts, three extensometer probes were also installed between the two rows of instrumented bolts in each gateroad. It is noted that the instrumented site at West Cliff Colliery did not have extensometric installations, due to unavailability of equipment at the time of instrumentation. The notations adopted for the bolts and extensometer probes in Mine 2 is explained in Figure 4.10. Field monitoring commenced soon after the installation in May 2000, and ended in October 2000, when the approaching longwall face overran the instrumented site.

Figure 4.10: Instrumentation naming convention at Tower Colliery, a) for bolts, and b) for extensometers.
4.3 INSTRUMENTATION

4.3.1 Instrumented Bolts

Bolts selected for instrumentation in both mines were identical except in length, which were similar to the normal bolting cycles in respective mines. At West Cliff Colliery (Mine 1), the production bolt was 2.1 m in length, whereas at Tower Colliery (Mine 2), the length of the bolt was 2.4 m. In both cases, the bolt diameter was 21.8 mm. The first step involved in the bolt instrumentation process was to cut two identical, diametrically opposite channels of 6 mm wide and 3 mm deep each along the bolt axis, leaving outer 100 mm intact as shown in Figures 4.11 and 4.12. The intact bolt configuration without any channel in the first 100 mm ensured the stability and integrity of the strain gauges and the corresponding wirings during the installation of bolt in the field. This has been the normal practice in bolt instrumentation (Signer and Jones, 1990b). Figure 4.13 shows a section of a bolt with engraved channels.

Once the channels were cut, they were smoothened with sand paper and wiped clean with alcohol solvents. A total of 18 strain gauges were mounted on each bolt (9 in each channel). The spacing between the strain gauges mounted on 2.1 m bolt, installed at West Cliff Colliery, was 200 mm (see Figure 4.11). On the other hand, the spacing on 2.4 m long bolts, installed at Tower Colliery, was kept at 250 mm intervals (see Figure 4.12). The variation in the strain gauge spacings on the bolts installed in two mines was necessary in order to accommodate the variation in bolt
lengths used. The slots were then filled with silicon gel to cover up the strain gauges, and allowed to harden for a week prior to installation in the field. Figure 4.14 shows a section of an instrumented bolt with strain gauges and wirings visible through the silicon cover.

Figure 4.11: Strain gauge and bolt layout for West Cliff Colliery (Mine 1).
Chapter 4: Field Investigations

Cross Section

Slot: 6 mm x 3.0 mm
Slot Area: < 10%

All Strain Gauges are 5 mm, 120 Ω, Gauge Factor 2.15

Reduction in bolt strength ≈ 10%

- Strain Gauges

Figure 4.12: Strain gauge and bolt layout for Tower Colliery (Mine 2).
Figure 4.13: Bolt segment showing channels.

Figure 4.14: A section of an instrumented bolt showing the strain gauge and wirings through the silicon gel.
4.3.2 Sonic Probe Extensometer

Bed separation and general ground deformation caused by face movement was monitored by using GEOKON 701 multi-point sonic probe extensometer. During monitoring, the extensometer probe (8 m long) was inserted in the guide tube located along the 38 mm diameter boreholes. Placed at a predetermined interval along the borehole were magnetic anchor points. A total of twenty reference magnets were housed in each borehole, and the position of each anchored magnet was picked up by the inserted extensometer probe.

The principle of the sonic probe relies on the magnetic properties of the probe material. An electrical pulse in the probe creates a magnetic field, which interacts with the field of a permanent magnet located in a borehole anchor. The change in the magnetic field generates, a torsional stress pulse in the probe material, which travels along the probe at the speed of sound. The arrival time of the stress pulse at the end of the probe is detected, and this is a reliable measure of magnet position. The position of each anchored magnet was measured and displayed directly on the GEOKON 701 readout box which has a least reading of 0.025 mm. Figure 4.15 shows various components of a sonic probe extensometer.
Figure 4.15: Various components of an extensometer, a) readout unit, b) probe, and c) photograph showing the process of taking readings in the underground.
4.3.3 Intrinsically Safe Strain Bridge Monitor

An intrinsically safe Strain Bridge Monitor (SBM), IS2000, was used for the underground measurement of strains developed in the instrumented bolts. SBM is set to operate with the more commonly used 120Ω strain gauges. By choice of appropriate bridge circuit, it was possible to measure the strain in a single gauge, two gauges or four gauges. The quarter bridge configuration was restricted to 120Ω strain gauges only. As the SBM had a fixed gauge factor setting of 2.00, the actual strain measurement indicated by the display could be calculated as follows:

\[ E = \frac{2V_d}{G} \quad \text{For quarter bridge configuration} \quad (4.1) \]

\[ E = \frac{V_d}{G} \quad \text{For half bridge configuration} \quad (4.2) \]

\[ E = \frac{V_d}{2G} \quad \text{For full bridge configuration} \quad (4.3) \]

where,

\( E \) = the mean actual strain measured by an active gauge,

\( V_d \) = the change in SBM reading, and

\( G \) = the gauge factor of the strain gauge.

Figure 4.16 shows a general view of the SBM, while taking readings in underground.
4.4 FIELD MONITORING AND DATA ANALYSIS

4.4.1 West Cliff Colliery

Site monitoring was conducted at regular intervals from the instrumented site. The primary horizontal stress around the region was estimated to be in the order of 16 MPa (Strata Control Technology, 1996), and its direction was as shown in Figure 4.5. The direction and the impact of horizontal stress were manifested clearly by excessive guttering at the left side of the C/T. Thus, the bolts at the left side of the cut through were subjected to excessive shear loading as compared to the bolts at the
right side of the cut-through. As can be seen in Figure 4.17, the bolts at the left side of the C/T experienced significantly higher load transfer in comparison to the bolts at the right side. The maximum load on the bolt at the left side was 220 kN, whereas the maximum load at the right side bolt was only 19.46 kN. Therefore, it may be concluded that the horizontal stress direction played an important role on the bolt load transfer mechanism. The redistribution of in-situ stress, because of heading development, has obviously caused differential impact on the reinforcement elements, such as bolts. The acute angle formed by the prevailing principal horizontal stress and the heading development direction caused the left side of the roadway to be under relatively higher shear loading as compared to the other side.

Figures 4.18 and 4.19 show the load transferred to the bolts and the corresponding shear stresses at the bolt/resin interface for both bolt types at the left side of the cut-through, respectively. No load build up comparison was possible for bolts installed at the middle of the cut-through, as the bolt number 2 of type I (see Figure 4.5) malfunctioned after a short period of installation. The shear stress at the bolt resin interface was calculated by using the following equation:

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

(4.4)

where,

\( \Delta \tau \) = average shear stress at the bolt-resin interface,

\( F_1 \) = axial force acting in the bolt at strain gauge position 1, calculated from strain gauge reading,
\( F_2 = \) axial force acting in the bolt at strain gauge position 2, calculated from strain gauge reading, 

\( d = \) bolt diameter, and 

\( l = \) distance between strain gauge position 1 and strain gauge position 2.

Figure 4.17: Load transferred on the type II bolts, West Cliff Colliery.
Figure 4.18: Load transferred on the bolts installed at the left side of the C/T, West Cliff Colliery.
Figure 4.18 shows the load transferred on the bolts, both type I and type II, installed at the left side of the cut-through. As observed from the figure, the axial load transfer on both type II and type I bolts was nearly of the same magnitude (Figures 4.18a and 4.18b), however, the shear stress sustained by the bolt/resin interface was slightly higher in the type II bolt as compared to the type I bolt (Figures 4.19a and 4.19b).
Figures 4.20 and 4.21 show the axial load transferred on the bolts and the corresponding shear stress on type I and type II bolts, installed at the right side of the cut-through, respectively. There is a considerable difference between the loads transferred in type I and type II bolts. Clearly, the axial load transferred on the type I bolts was much higher than that of type II bolts (see Figures 4.20a and 4.20b) due to the impact of both magnitude and direction of prevailing principal horizontal stress in the instrumented site. The shear stress sustained by the bolt/resin interface was also higher in the type I bolts as compared to the type II bolts (see Figure 4.21a and 4.21b).

Figure 4.20: Load transferred on bolts installed at the right side of the C/T, West Cliff Colliery.
Figure 4.21: Shear stress developed at the bolt/resin interface of the bolts installed at the right side of the C/T, West Cliff Colliery.

The examination of the load transferred and subsequent shear stress build up on the bolts at the right side have revealed a different picture in comparison with the bolts installed at the left side of the C/T. Clearly, the level of load build up on the bolts installed at the left side of the C/T was influenced by the presence and the direction of high horizontal stress.
4.4.2 Tower Colliery

Following the installation of instrumented bolts and extensometers, the site was monitored at regular intervals. The primary horizontal stress around the region was estimated to be 25 MPa (Strata Control Technology, 1996), and the direction was nearly parallel to the axis of the gateroads as shown earlier in Figure 4.9. The regional stress pattern indicated that, the direction of the principal horizontal stress should be near perpendicular to the axis of the gateroads. However, the presence of a major dyke at the outbye of the instrumented sites caused reorientation of the prevailing horizontal stress direction (see Figure 4.8). Severe guttering and deteriorations in general roof conditions were observed up to the location of the dyke, and beyond that, there was a relative improvement in ground conditions (including the instrumentation site) with minor guttering observed along the left side of both the access roads as shown in Figure 4.22a. No guttering was observed on the other (right) side of both the gateroads (see Figure 4.22b). Thus, it can be inferred that the bolts on either side of the gateroads were subjected to differential shear loading, with the left side being subjected to sightly higher loading than that on the right side.

Figure 4.22: Roof condition in the gate roads at Tower Colliery, a) left side, and b) right side.
4.4.2.1 Load transfer during the panel development and the longwall retreating phases

Figures 4.23 and 4.24 show the overall load transferred on the type II bolts during panel development and subsequent longwall retreating phases, installed at the left side of the travelling road and the belt road (numbered as TRA1 and BRA1 in Figure 4.9), respectively. As expected, the load transferred on the bolts during the panel development stage was relatively low as compared to the longwall retreating phase in both the travelling and belt roads. The load developed up due to the front abutment pressure of retreating longwall face was, on the average, 5 to 8 times higher than that of panel development phase (see Figures 4.23 and 4.24).

Figure 4.23: Load transferred on the bolt TRA1, during the panel development and longwall retreating phases, Tower Colliery.
Figure 4.24: Load transferred on the bolt BRA1, during the panel development and longwall retreating phases, Tower Colliery.

The maximum load transferred to the bolt BRJ2 at a particular face position, during panel development and longwall retreating phases, is shown in Figure 4.25. The load transferred during the panel development stage became constant within 40m advance of development headings, away from the instrumented sites. However, the load build up due to the front abutment pressure of the approaching longwall face began to increase significantly, when the distance was 150m from the test sites.
Figure 4.25: Maximum load transferred on the bolt BRJ2, for a particular face position, Tower Colliery.

**4.4.2.2 Load transfer in the belt and travelling road**

Figure 4.26 shows the load transferred on the bolts BRJ2 and TRJ2 (both type I) respectively, during the panel development stage. The corresponding load transferred on the same bolts, during the longwall retreatting phase, is shown in Figure 4.27. No significant changes in load transfer were observed, either during the panel development (Figure 4.26) or during the longwall retreated (Figure 4.27) phases. However, the load transferred on the belt road, in general, was slightly greater as compared to the travelling road during both panel development and longwall retreating phases. The differential response by the instrumented bolts in the gate roads was expected because of the relative positions of the gate roads with respect to the mined longwall panel as shown in Figure 4.8.
Figure 4.26: Load transferred on the bolts TRJ2 and BRJ2 during the panel development phase, Tower Colliery.
Figure 4.27: Load transferred on the bolts TRJ2 and BRJ2 during the longwall retreating phase, Tower Colliery.

Figure 4.28 shows the maximum load transferred on to BRJ2 and TRJ2 for a particular face position, during the panel development and the longwall retreating phases. In both gateroads, the load on the bolts started to build up immediately after their installation during the panel development stage, and then became constant when the heading development face was about 50m away from the test sites (see darker lines in Figures 4.28a and 4.28b). During the longwall retreating phase, however, the impact of front abutment pressure was observed (by sharply increasing load), when
the approaching longwall face was around 60m away from the test site in case of travelling road (Figure 4.28a).

Figure 4.28: Maximum load transferred on the bolts TRJ2 and BRJ2, for a particular face position, Tower Colliery.
In case of belt road, the same was observed when the approaching longwall face was about 150m away from the test site (Figure 4.28b). Thus, the impact of longwall face movement was more prominent in case of belt road as compared to the travelling road. It is noted that, the travelling road was adjacent to the solid longwall block (longwall panel 19, yet to be mined), whereas the belt road was driven in a relatively disturbed zone between the present longwall face and the travelling road, thereby influencing the load transfer mechanism.

4.4.2.3 Behaviour of strata deformation

Figures 4.29 - 4.31 show the overall roof deformation recorded from the extensometry readings in the travelling road. As expected, the maximum deformation was recorded in the middle of the road (Figure 4.30) because of the prevailing near parallel principal horizontal stress direction, and was aggravated by the deadweight of the separated sagging roof. The amount of roof deformation recorded at the left side of the gateroad (Figure 4.29) was relatively small as compared to the middle section (Figure 4.30), but was greater than that on the right side (Figure 4.31) of the roadway. Also, the acute angle between the horizontal stress direction and the axis of the gateroads caused some shearing in the immediate roof at the left side of the gateroads, while the other side of the gateroads was relatively stable, with negligible amount of strata deformation (Figure 4.31). No significant strata deformation was observed at a horizon level of more than 3m from the roof level. Thus, it may be suggested that, in addition to the regular bolt pattern at Tower Colliery, occasional use of longer secondary reinforcement units (e.g. cable bolts) may be required for
effective heading stabilisation. However, it is difficult to suggest similar strata reinforcement pattern at outbye side of the dyke, because of varying stress conditions.

Figure 4.29: Strata deformation recorded in the extensometer TR1, Tower Colliery.
Figure 4.30: Strata deformation recorded in the extensometer TR2, Tower Colliery.
Figure 4.31: Strata deformation recorded in the extensometer TR3, Tower Colliery.

Figure 4.32 shows the maximum deformation (recorded from TR1) with respect to the face distance from the test site, during the panel development and the longwall retreating phases. As can be seen from the figure, a negligible amount of roof deformation was recorded during the panel development stage. The roof deformation began to build up, when the approaching longwall face was about 50 m from the instrumented site. The pattern of deformation was similar to the pattern of load build up on the bolts as discussed in Section 4.4.2.2.
Figure 4.32: Maximum deformation recorded in the extensometer TR1, for a particular face position, Tower Colliery.

Figure 4.33 shows the three-dimensional surface profile of the maximum deformations recorded by the extensometers in the travelling road, for a particular face position during the longwall retreating phase. As discussed before, the maximum deformation was found to occur near the middle section of the roadway, with significant deformation occurring when the approaching longwall face was less than 60m from the instrumented site.
Figure 4.33: Three-dimensional surface generated from the maximum strata deformations recorded in the travelling road, Tower Colliery.

4.4.2.4 Impact of strata deformation on the load transfer in the bolts

Figure 4.34 shows the relative displacement between the magnetic reference anchors along the borehole, for the extensometer BR2, installed in the middle of the belt road. The relative movement was initiated mainly at two levels i.e. at 600 mm and at 2300 mm levels above the roof. The load transferred on the bolt BRA2 (installed close to extensometer BR2) is shown in Figure 4.35. The maximum load transferred on to the bolt at 600 mm above the roof was in close agreement with the location of the maximum strata deformation. Thus, there exists a direct relationship between the load transfer in the bolt and the relative strata deformation at any particular horizon.
No comparison between the strata deformation and the load build up on the bolt could be drawn at 2300 mm level, as the position of the last strain gauge was at 2200 mm above the roof level.

Figure 4.34: Relative displacements between the two consecutive extensometer probes in BR2, Tower Colliery.
Chapter 4: Field Investigations

Figure 4.35: Load transferred on the bolt BRA2, during the longwall retreating phase, Tower Colliery.

The maximum strata deformation recorded on TR2, and the maximum load transferred on the bolt TRA2 due to front abutment pressure, with respect to the approaching longwall face position are shown in Figures 4.36 and 4.37, respectively. Similar trends in the maximum strata deformation and the maximum load transferred on the bolt were observed, both showing significant increase when the approaching face was about 60m from the test site.

Figure 4.36: Maximum deformation recorded in the extensometer TR2, for a particular face position, Tower Colliery.
4.4.2.5 Comparison of load transfer in type I and type II bolts

Figure 4.38 shows the load transferred on the bolts TRJ1, TRJ2 and TRJ3, installed at the left, middle and the right side of the travelling road. The maximum load recorded on the above bolts was 39 kN, 97.6 kN and 33.5 kN, respectively. The bolt at the left side was subjected to relatively higher load as compared to the bolts at the right side, which may have been due to the influence of the orientation of principal horizontal stress, striking the gateroads with an acute angle from the left side (Figure 4.9). When compared with other bolts, the bolt at the middle of the road recorded the maximum value because of the dominant role of excessive strata deformation in the middle of the gateroads.
Figure 4.38: Load transferred on the type I bolts, installed in the travelling road, Tower Colliery.
Figure 4.39 shows the pattern of load transferred on the type I (BRJ1) and type II (BRA1) bolts, installed at the left side of the belt road. The load transferred on the type I bolts was relatively smaller as compared to the type II bolts. The maximum load transferred on BRJ1 and BRA1 was 41.7 kN and 84.6 kN, respectively. The corresponding shear stress developed at the bolt/resin interface for both bolts is shown in Figure 4.40. The comparative values observed from the shear stress profiles of BRJ1 and BRA1 suggests that, the bolt type II offered better load transfer characteristics when subjected to higher shear loading, caused by the influence of the horizontal stress.

![Graph of load transferred on type I (BRJ1) and type II (BRA1) bolts](image)

Figure 4.39: Load transferred on the type I and type II bolts, installed at the left side of the belt road, Tower Colliery.
Chapter 4: Field Investigations

Figure 4.40: Shear stress developed at the bolt/resin interface of the type I and type II bolts, installed at the left side of the belt road, Tower Colliery.

The neutral point (zero shear stress position), as defined by Tao and Chen (1983), for both BRJ1 and BRA1 occurred at around 500 mm above the roof level. Accordingly, the pickup length (from roof level to the neutral point) and the anchor length (from neutral point to the end of the bolt) were calculated to be 0.5 m and 1.8 m, respectively. Thus, it can be inferred that, the location of the neutral point is independent of the bolt type.
Figure 4.41 shows the load transferred in type I (BRJ3) and type II (BRA3) bolts, installed at the right side of the belt road. As expected, the load transferred in type I bolt (38.1 kN) was relatively higher as compared to the type II bolt (18.6 kN). Because of the lower influence of the horizontal stress on the bolts, the shear stress developed at the bolt/resin interface in type I bolt was relatively higher than in type II bolt (see Figure 4.42), thus, reconfirming the superior load transfer characteristics of the type I bolts under lower levels of prevailing horizontal stress.

Figure 4.41: Load transferred on the type I and type II bolts, installed at the right side of the belt road, Tower Colliery.
Figure 4.42: Shear stress developed at the bolt/resin interface of the type I and type II bolts, installed at the right side of the belt road, Tower Colliery.

The neutral point for both BRJ3 and BRA3 was found to be at around 820 mm above the roof level. The neutral point appears to be dependent on the stress condition at the installed bolt location and the strata deformation behaviour. This was evidenced by the different neutral point locations (for the same type of bolt) of 500mm on the left side and 820mm on the right side of the gateroad respectively.
On the other hand, the pattern and the magnitude of the load transferred on the type I (TRJ2) and type II (TRA2) bolts, installed at the middle of the gateroad, is shown in Figure 4.43. The near parallelism of the direction of the principal horizontal stress to the axis of the gateroads caused insignificant shear loading on the bolts installed at the middle of the gateroads. The strata deformation due to the dead weight of the immediate roof, which affected both bolt types equally, caused almost equal levels of load transfer on both type of bolts. This is clearly demonstrated by the pattern of load distribution shown in Figure 4.44, where-by the load profiles for both bolt types I and II are similar during the entire longwall retreating phase. Similarly, the shear stresses developed at the bolt/resin interface of these two bolts also showed identical behaviour (see Figure 4.45).

Figure 4.43: Load transferred on the type I and type II bolts, installed at the middle of the travelling road, Tower Colliery.
Figure 4.44: Maximum load transferred on type I and type II bolts for a particular face position, installed at the middle of the travelling road, Tower Colliery.

Figure 4.45: Shear stress developed at the bolt/resin interface of the type I and type II bolts, installed at the middle of the travelling road, Tower Colliery.
The three-dimensional surface profiles generated from the maximum load transferred with respect to the face distance from the instrumented site, on type I and type II bolts, installed in the travelling road is shown in Figure 4.46. Both surfaces indicated that the maximum load build up occurred at the middle of the gateroad. Some noticeable amount of load build up occurred when the retreating longwall face was at a distance of less than 60 m from the instrumented site in both bolt types.

Figure 4.46: Three-dimensional surface generated from the maximum load transferred on type I and type II bolts, installed in the travelling road, Tower Colliery.
4.5 BOLT SELECTION GUIDELINES

The laboratory experiments discussed in Chapter 3 suggested that the type I bolts offered higher shear resistance as compared to the type II bolts at low normal stress levels (confining stress condition in the field) on the bolt/resin interface. At higher normal stress levels, the type II bolts offered marginally higher shear resistance as compared to the type I bolts. Similar phenomenon was also observed from the field investigations. When the bolt was subjected to high shear loading in the field, the shear stress sustained by the type II bolt was marginally higher than the type I bolt; whereas when the bolt was subjected to low shear loading, the shear stress sustained by the type I bolts was significantly greater than the type II bolts. Thus, the degree of shear stress sustained by the bolt was influenced by the level of the applied initial normal stress on the bolt/resin interface or the confining pressure (in the field) and the surface profile configuration of the bolt. Accordingly, bolts with deeper and wider rib spacing should be used in section of the roadways subjected to the influence of low horizontal stress, whereas, bolts with shallower and narrower rib spacing should be used in areas under the influence of high horizontal stress (as evidenced by the excessive roof guttering).
4.6 CONCLUSIONS

The field investigations conducted in two coal mines in the Illawarra region of NSW, Australia, revealed the following aspects of the load transfer mechanism.

- The load transfer on the bolt was influenced by; a) the confining stress condition, b) the extent of strata deformation, and c) the surface profile roughness of the bolts.
- The load transferred on the bolts, during the longwall retreating phase, was relatively greater than that of panel development phase.
- The face movement did not influence the load transferred on the bolts, when the development face moved beyond 50m away, or the approaching longwall face position was 150m away from the test sites.
- There was no significant variation of the load transfer magnitude on the bolts in either gateroads (belt road and travelling road) during the panel development stage.
- The influence of front abutment pressure build up on the gateroads appears at different face positions. The load build up on the bolts in the belt road occurs when the longwall face is less than 150m from the test site, whereas, the same build up on the travelling road starts when the face position is less than 60m.
- The maximum strata deformation was found to occur in the middle of the gateroads.
- The maximum load transferred along the bolts was found to occur at the level of maximum deformation.
- The neutral point was found to be independent of the type of bolt used, and its value changes with varying stress conditions and strata deformation characteristics.

- The field study showed that, under the low influence of horizontal stress (both in magnitude and the direction), the type I bolt offered higher shear resistance, whereas under high influence of horizontal stress, the type II bolt offered better shear resistance at the bolt/resin interface. Such findings were also observed in the laboratory studies, and they provide a useful guide for future selection of appropriate bolt types for given stress conditions.
Chapter 5

SHEAR BEHAVIOUR OF ROCK JOINTS

5.1 INTRODUCTION

Joints in rock can be open or closed. Closed joints may contain infill, which is either loose or cemented. The nature of joint fillings, thus has considerable influence on the strength properties of the rock mass. Rocks with weaker infill joints are generally weaker than the same type of rocks with well-cemented joints.

Irrespective of the fillings, rock bolting is widely used for strengthening rock joints, and the application of bolts has been the subject of considerable research during the past several decades. Much of the studies were concerned with the effective application of bolts with respect to the bolt orientation with the joint plane. The joint orientation is of significant concern in tunnel construction and underground coal mining heading development, particularly in weak rock stratification. This chapter, is therefore, concerned with the critical review of the role of bolting in jointed rock mass stabilization.
5.2 REVIEW OF UNFILLED JOINTS

In recent years there has been a growing interest on the study of the shear behaviour of rock joints under Constant Normal Stiffness condition, instead of the traditional method of testing under Constant Normal Load conditions. The pioneering work of Patton (1966), based on a series of tests on regular saw-tooth shaped artificial joints, was carried out under CNL condition, and test results closely fitted to a bi-linear shear strength envelope as described in Figure 5.1. Accordingly:

\[ \tau_p = \sigma_n \times \tan(\phi_b + i) \]

for asperity sliding \hspace{1cm} (5.1)

\[ \tau_p = c + \sigma_n \times \tan(\phi_b) \]

for asperity shearing \hspace{1cm} (5.2)

where,

\[ \tau_p = \text{peak shear stress}, \]
\[ \sigma_n = \text{normal stress}, \]
\[ \phi_b = \text{basic friction angle}, \]
\[ c = \text{cohesion intercept}, \] and \[ i = \text{initial asperity angle}. \]

According to Patton (1966), the sliding of asperities takes place under low normal stress, and beyond a certain normal stress value, shearing through asperities may also result. It has been observed that the peak shear strength predicted by Patton’s model,
especially at low to medium normal stress conditions, generally overestimates the actual shear strength.

Barton (1973) also introduced a non-linear strength envelope for non-planar rock joints, tested under Constant Normal Load (CNL) condition, and was given by:

$$\left( \frac{\tau_p}{\sigma_n} \right) = \tan \left[ \phi_b + JRC \times \log_{10} \left( \frac{\sigma_c}{\sigma_n} \right) \right]$$  \hspace{1cm} (5.3)

where,

$$\phi_b = \phi - (d_n + s_n),$$

$$\phi = \text{friction angle},$$

$$d_n = \text{peak dilation angle which decreases with the increase in normal stress},$$
s_n = angle due to shearing of asperities which increases with the increase of the normal stress as more surface degradation occurs,

JRC = Joint Roughness Coefficient, and

σ_c = uniaxial compression strength.

The JRC-JCS model developed by Barton (1973, 1976) and Barton and Bandis (1982, 1990) has almost dominated the practice of rock joint engineering and is now considered to be a *de facto* international standard. However, despite this recognition, there remains a concern because of the highly empirical basis of the model, which may lead to more conservative design solutions. Others engaged in similar research work include Ladanyi and Archambault (1970), Barton (1973, 1976, 1986), Hoek (1977, 1983, 1990), Hoek and Brown (1980), Bandis et al. (1983), Hencer and Richards (1989), Kulatilake (1992), Kulatilake et al. (1993), Saeb and Amadei (1992), and Brady and Brown (1993), where all these researchers conducted their studies under CNL or zero stiffness condition.

The joint research under Constant Normal Stiffness condition is a relatively new concept. The importance of CNS condition, with regard to the shear strength of any joint plane, has been described in Chapter 2. The shear strength of non-planar joint increases due to application of external normal stiffness, K_n, which allows a joint to shear under restricted dilation. As a result, the peak shear stress of joints would be higher under CNS condition in comparison to that of CNL condition. Leichnitz (1985) was the first to report on the laboratory studies of rock joints under CNS condition, and he found that both the shear force and the dilation were functions of
the normal force and shear displacement. Since then, similar findings have been reported by Benmokerane and Ballivy (1989), Van Sint Jan (1990), Ohnishi and Dharmaratne (1990), Skinas et al. (1990), Haberfield & Johnstone (1994), and Haberfield and Seidel (1998).

Johnstone & Lam (1989) developed an analytical method for the shear resistance of concrete/rock interface under CNS condition. By assuming the penetration of microasperities of concrete into the rock surface, when the contact normal stress exceeds the uniaxial compressive strength (see Figure 5.2), they formulated the following equation for the mobilised cohesion, $c_m$:

\[
    c_m = \frac{c_{sl}}{\pi} \cos^{-1} \left(1 - \frac{2\sigma_n}{q_u}\right)
\]  

(5.4)

where,

- $c_{sl}$ = cohesion of rock for asperity sliding,
- $\sigma_n$ = actual contact normal stress, and
- $q_u$ = uniaxial compressive strength.
In another paper, Haberfield and Seidel (1998) have extended the CNS technique to include soft rock/rock and concrete/rock joints. The following theoretical model was proposed, based on the careful observation of laboratory direct shear testing on joints containing two-dimensional roughness:

\[
\tau = \frac{1}{A} \sum_{j=1}^{n} a_j \sigma_{n_j} \tan(\phi_b + i_j)
\]  

(5.5)
where,

\[ \tau = \text{shear strength of joint}, \]
\[ A = \text{total joint contact area}, \]
\[ n = \text{number of asperities}, \]
\[ a_j = \text{contact areas of the individual asperities}, \]
\[ \sigma_{nj} = \text{normal stresses acting on the individual asperities}, \]
\[ \phi_b = \text{friction angle, and} \]
\[ i_j = \text{variable asperity angles}. \]

The processes modelled also included asperity sliding, asperity shearing, post-peak behaviour, asperity deformation, and distribution stresses on the joint interface. Model predictions were compared with the laboratory results, and although the model predictions compared favourably with laboratory test results for concrete/rock joints, nevertheless, the strength values were over-predicted when applied to rock/rock joints.

### 5.3 REVIEW OF UNFILLED REINFORCED JOINTS

The shear behaviour of reinforced joints has remained the focus of research in geotechnical engineering particularly in; a) the contribution of bolts to the shear strength of reinforced joints, b) the deformation behaviour, and c) the stability of reinforced joints. Figure 5.3 shows typical deformation behaviour of a reinforced joint under shear loading.
The study of reinforced joints was first initiated by Bjurstrom (1974). Bjurstrom conducted a series of shear tests, both in the laboratory and in the field on granite specimens reinforced by fully grouted steel bolts, to determine the influence of various factors affecting the shear strength of reinforced rock joints. He found that both the rock strength and the bolt orientation with respect to the joint plane influenced the strength of the reinforced joint (see Figure 5.4). Bjurstrom reported that, the bolt failure characteristics were dependent upon the angle between the bolt and the joint planes, and the bolt failure in tension occurred when the angle was less than $35^\circ$. He also suggested that, the shear strength of such rock was dependent on the following three parameters.
(i) Shear resistance due to reinforcement effect:

\[ T_b = P(Cose\theta + \mu Sin\theta) \]  \hspace{1cm} (5.6)

where,

- \( P = \) bolt tension,
- \( \theta = \) angle between the bolt and the joint,
- \( \mu = \tan(\phi), \) and
- \( \phi = \) angle of internal friction.

(ii) Shear resistance due to dowel effect:

\[ T_d = 0.67 d^2 (\sigma_s \sigma_c)^{0.5} \]  \hspace{1cm} (5.7)

where,

- \( d = \) bolt diameter,
\( \sigma_s \) = bolt yield strength, and
\( \sigma_c \) = uniaxial compressive strength of rock.

(iii) Shear resistance due to friction of joint itself:

\[
\tau_j = A_j \sigma_n \tan \phi_j \quad (5.8)
\]

where,
\( A_j \) = joint area,
\( \sigma_n \) = normal stress on joint, and
\( \phi_j \) = joint friction angle.

The total resistance offered by a bolt was given by the summation of all the above three components as shown in Figure 5.5.

Hass (1981) conducted laboratory studies on limestone specimens with an artificially cut joint, reinforced by full scale rock bolts of various types, which intersected the shear plane at different orientations as shown in Figure 5.6. It was concluded that, the orientation of rock bolts relative to the shear plane had a pronounced effect on the shear resistance offered by a bolt. He suggested that the bolts would act more effectively when they are inclined at an acute angle to the shear surface than the opposite direction, as they tend to elongate as shearing progress. The shear strength of a bolted joint was found to be the sum of the bolt contribution and the friction
resulting from the normal stress on the shear plane. The contribution due to shearing of asperities was not considered in the model.

Figure 5.5: Components of shear resistance offered by a bolt (after Bjurstrom, 1974).

Figure 5.6: Arrangement for bolt shear testing (after Hass, 1981).
Based on several laboratory and field results, Spang and Egger (1990) provided a detailed analysis of the behaviour of fully grouted bolts in jointed rocks. The results were used to explain the mode of action of fully bonded, untensioned rock bolts in stratified or jointed rock mass. A three dimensional model was implemented using the Finite Element Method (FEM) to support the laboratory findings. Three distinguished stages of behaviour (elastic, yielding, and plastic) were observed during shearing of the grouted joints. A mathematical model was proposed to determine the shear resistance of bolted rocks:

\[ T_o = P_t [1.55 + 0.011 \sigma_c^{1.07} \sin^2(\alpha + i)] \sigma^{-0.14} (0.85 + 0.45 \tan \phi) \]

(5.9)

where,

- \( T_o \) = maximum shear resistance of grouted rock,
- \( P_t \) = maximum tensile load of the bolt,
- \( \sigma_c \) = uniaxial compressive strength of rock,
- \( \alpha \) = inclination of bolt with respect to joint plane,
- \( \phi \) = angle of internal friction, and
- \( i \) = asperity angle.

The above equation is valid for rocks with the uniaxial compressive strength within the range of 10 to 70 MPa and for angles between the bolt and the joint surface within the range of 60 to 90 degrees.
Aydan and Kawamoto (1992) used stability analysis method for slopes and underground openings under various loading conditions against flexural toppling failure. A method was suggested to take into account, the reinforcement effect of fully grouted rock bolts. It was reported that the installation angle of rock bolts, $\theta$, must be between $0^\circ$ and $+90^\circ$ in order to make their effective use. Theoretically, bolts should be most effective when the installed angle $\theta$ is equal to $90^\circ - \phi$ ($\phi$ = joint friction angle), as shown in Figure 5.7. The proposed analytical model was validated by the laboratory tests under controlled environment. The reinforcing effect of a rock bolt on the shear resistance of a discontinuity plane was expressed in the following form:

$$T_b = A_b \sigma_b \left(1 + \frac{1}{2} \tan \phi \cdot \sin \theta \right)$$  \hspace{1cm} (5.10)

where,

- $T_b$ = shear resistance due to bar,
- $A_b$ = cross section of bar,
- $\sigma_b$ = magnitude of stress in the bar in the direction of shearing,
- $\phi$ = friction angle of discontinuity, and
- $\theta$ = angle between bolt axis and discontinuity.

The output from the analytical model was compared favourably with those from the numerical model using FEM as shown in Figure 5.8. In case of a bolt placed perpendicular to the joint plane, Aydan and Kawamoto (1992) concluded that the bolt
contribution was independent of the joint friction angle, which is not the case in reality, due to stress rotation upon shearing.

Figure 5.7: Effect of bolt inclination on the bolt shear resistance (after Aydan and Kwamoto, 1992).
Ferrero (1995) proposed a shear strength model for reinforced rock joints, based on the numerical and laboratory studies on large size shear blocks. Ferrero suggested that, the overall strength of the reinforced joint could be attributed to the combination of both dowel effect and the incremental axial force due to the bar deformation, as shown in Figure 5.9. The proposed analytical model was expressed by:

\[ F = T_r \cos \alpha - Q \sin \alpha - (T_r \sin \alpha - Q \cos \alpha) \tan \phi \]  

(5.11)
where,

\[ F = \text{global reinforced joint resistance}, \]
\[ T_r = \text{tensile load in bolt}, \]
\[ Q = \text{force due to dowel effect}, \]
\[ \alpha = \text{angle between the joint and the dowel axis, and} \]
\[ \phi = \text{joint friction angle}. \]

The above model is mainly applicable to the bolts installed perpendicular to the joint plane in stratified horizontal bedding planes.

Figure 5.9: Resistance mechanism of a reinforced rock joint (after Ferrero, 1995).

Windsor (1997) proposed a method for the design of reinforcement for excavation in jointed rocks using the systems approach. The method was based on identifying and stabilising blocks of rock that may form at the mine excavation boundary, which is the core of block theory. According to Windsor (1997), the load transfer concept consisted of three basic mechanisms:
• Rock movement and load transfer from an unstable zone to the reinforcing element.

• Transfer of load from the unstable region to a stable interior region via the element.

• Transfer of the reinforcing element load to the stable rock mass.

The above mechanisms were supported by both the mathematical and numerical models, but without any experimental or field verification.

Pellet and Boulon (1998) proposed an analytical model for predicting the contribution of a bolt to the shear strength of rock joints. The model took into account, the interaction of the axial and shear forces mobilised in the bolt, as well as its large plastic deformation occurring during the loading process. The shape of the stressed bolt and the failure envelope for both elastic and plastic deformations are shown in Figures 5.10 and 5.11, respectively. The proposed failure criteria is in the form of:

\[
Q_{of} = \frac{\pi D_b^2}{8} \sigma_{ec} \sqrt{1 - \frac{16 N_{of}}{N_{of}^2}}
\]

where,

\[
Q_{of} = \text{shear force acting on the bolt at the joint plane at the point of failure of the bolt},
\]
$D_b = \text{bolt diameter,}$

$\sigma_{oe} = \text{failure stress of bolt, and}$

$N_{of} = \text{axial force acting on the bolt at the joint plane at the point of failure of the bolt.}$

Figure 5.10: Force components and deformation of a bolt, a) in elastic zone, and b) in plastic zone (after Pellet and Boulon, 1998).
The axial force acting on the bolt is difficult to determine without the relevant instrumentation, and the intermediate steps necessary to calculate the failure strength are also complicated. In addition, the model assumes a constant rock reaction during shearing process, irrespective of the applied normal stress and the bolt material.

5.4 REVIEW OF INFILLED JOINTS

As described earlier, the presence of joints in the rock mass plays an important role on the overall shear and deformability behaviour of rocks, in addition to in-situ stress
Chapter 5: Shear Behaviour of Rock Joints

and hydro-geological conditions. The behaviour of a joint is governed by the fact whether the joint is open or closed with infill material. Typical infill materials existing within the joint interfaces may be divided to; a) loose material from the surface e.g. sand, clay etc., b) deposition by ground water flow, c) loose material from tectonically crushed rock, and d) products of decomposition and weathering. The thickness of infill material can vary from a fraction of a micron to several centimetres, depending on the situation in which the infill was deposited. For smooth joints, the thickness of the infill material does not play a significant role, and for rough joints the interaction between the infill/asperity and asperity/asperity could occur, depending on the geometry of the asperities and the infill thickness. When the infill thickness \( t \) is sufficiently large, for instance, more than twice the asperity height \( a \), the shear behaviour would be governed by the characteristics of infill material alone. Clearly, the \( t/a \) ratio appears to play an important role in the behaviour of infilled joints.

During the last three decades, various researchers have extensively evaluated, the laboratory shear strength parameters of both natural and artificial infilled rock joints (e.g. Goodman, 1970; Kanji, 1974; Ladanyi & Archambault, 1977; Lama, 1978; Barla et al., 1985; Bertacchi et al., 1986; Pereira, 1990; Phien-wej et al., 1990, and de Toledo & de Freitas, 1993). Parameters considered by various researchers affecting the infill joint behaviour include the joint type, infill thickness and characteristics, drainage conditions, boundary conditions, infill-rock interaction, and the system stiffness.
Goodman's (1970) research work on saw-tooth shaped joints, filled with crushed mica, found that, the shear strength of the joint was greater than that of the infill alone up to a t/a ratio of 1.25 as shown in Figure 5.12. Similar findings were reported also by Ladanyi and Archambault (1977), from their studies on concrete blocks with kaolin clay infill (Figure 5.13). They also found that, the value of the shear strength increased with the increasing asperity angle, and with the decreasing t/a ratios.

![Figure 5.12: Shear strength of mica infilled joint (after Goodman, 1970).](image)

Research studies on sand and clay infilled joints of limestone, sandstone and marl undertaken by Tulinov and Molokov (1971) found that, a thin layer of sand did not have any significant influence on the frictional behaviour of hard rocks. In contrast, the tests on soft rocks showed the opposite result. A study by Lama (1978) indicated that the strength of infill joint approached the strength of infill material when the t/a ratio exceeded the critical value of unity. In some cases, the joint shear strength dropped to that of infill, with a t/a ratio of less than unity. Kutter and Ruttenberg
(1979) study showed that, the shear strength of the clay infilled joint increased significantly with increasing surface roughness, while for sand filled joint, the increase was insignificant. Phien-wej et al (1990) study on a series of tests on saw-tooth shape gypsum samples revealed that, the strength of the joint becomes equal to that of the infill material, when the t/a ratio approaches two (Figure 5.14).

Figure 5.13: Shear strength of kaolin infilled joint (after Ladanyi and Archambault, 1977).

Papalingas et al. (1993) reported a detailed shear tests on plaster-cement modelled joint filled with each of kaolin, marble dust and pulverised fuel ash. He found that, the shear strength of joints containing kaolin became constant at a t/a ratio of about 0.6, and for marble dust or fuel ash, the values were between 1.25 and 1.50. There was a marked reduction in shear strength, with the addition of a thin layer of infill material. As can be seen from Figure 5.15 that, any further increase in the t/a values beyond 1.5 it would not result in any significant increase in the peak shear strength.
This is clearly evident by the asymptotic shape of the curves to the t/a axis. De Toledo and de Freitas (1993) study on ring shear tests on toothed Penrith sandstone and Gault clay exhibited two different levels of peak shear stress (the soil peak and the rock peak) along the joint shear stress path. The reduction in soil peak shear stress was observed up to a t/a ratio of unity, and beyond that, it tapers off and becomes parallel to the displacement axis. The rock peak shear stress, on the other hand, was the same, regardless of the level of consolidation pressure applied on the infill material.

Figure 5.14: Variation of shear strength with t/a ratio (after Phien-Wej et al., 1990).
Figure 5.15: Effect of $t/a$ ratio on shear strength of infilled joints (after Papalingas et al., 1993).

The study by De Toledo and de Freitas (1993) on CNL or zero stiffness condition indicated that different results would be obtained if the normal stiffness of the apparatus were changed. In contrast, testing by Cheng et al. (1996) on Johnston paste (mixture of mudstone powder, cement and water) infilled concrete/rock interfaces under CNS condition, showed that, unlike in CNL conditions, the presence of a very thin smear zone ($t<1$ mm) did not significantly reduce the shear strength of joints. Almost the same strength values were observed for dry cast joints with and without infill, and up to infill thickness of 2 mm. This conclusion, however, contradicted the findings from CNL testing reported by others (Goodman, 1970; Lama, 1978; Papalingas et al., 1993).
Indraratna et al. (1999) conducted CNS tests on two different asperity angles of $9.5^0$ and $18.5^0$, with joints infilled with soft clays described as Type I and Type II joints, respectively. The samples were tested under various initial normal stress values, ranging between 0.3 MPa and 1.10 MPa, respectively, and at a constant normal stiffness of 8.5 kN/mm. It was found that, the presence of infill in joints contributed to significant reduction in both the shear strength and the friction angle (Figure 5.16). The results also showed that, the effect of asperities on the shear strength was significant, with up to a $t/a$ ratio of 1.4. The infill alone had a controlling influence on the shear behaviour beyond the stated critical ratios. The drop in peak shear stress under CNS condition was modelled by a hyperbolic relationship. A Fourier transform method was also introduced to predict both the dilation and the shear strength of infilled joints under CNS condition.

To the best of author’s knowledge, no suitable literature was available at the time of writing this thesis to report on the shear behaviour of infilled bolted joints under CNS condition.
Chapter 5: Shear Behaviour of Rock Joints

Figure 5.16: Effect of infill on strength envelope, a) joint with asperity angle 9.5°, and b) joint with asperity angle 18.5° (after Indraratna et al., 1999).
5.5 CONCLUDING REMARKS

It can be inferred, that the critical review reported in this chapter leads to the following conclusions:

- Most unfilled joint research has been conducted under CNL condition.
- All the reinforced joint research (reported in literature) was conducted under CNL condition.
- Only a handful of research on both unfilled and infilled joints has been reported recently under CNS condition.
- No research work has been reported on infilled bolted joints.

Having recognised the relevance and the importance of evaluating the shear behaviour of bolted joints under Constant Normal Stiffness condition; there is a definite need for understanding the shear behaviour of both bolted and non-bolted joints. In this thesis, a critical examination of the shear behaviour of unfilled and infilled joints containing fully grouted bolts is reported, together with a mathematical model to predict the shear resistance of both unfilled and infilled bolted joints using Fourier analysis.
Chapter 6

SHEAR BEHAVIOUR OF UNFILLED BOLTED JOINTS

6.1 INTRODUCTION

This chapter deals with the shear behaviour of unfilled bolted joints under Constant Normal Stiffness conditions. The laboratory investigation was carried out using specially cast saw-toothed blocks bolted with 3 mm diameter rods. The tests were conducted in a CNS testing machine as described in Chapter 3. The test results were critically analysed and supplemented with a mathematical model later (Chapter 8).

6.2 LABORATORY INVESTIGATION

6.2.1 Selection of Model Material

The idealised joints were prepared from gypsum plaster (CaSO₄·H₂O hemihydrate, 98%). Gypsum was selected for the model material mainly because it was easy to cast, universally available and inexpensive, as well as widely used by geotechnical researchers. Gypsum plaster can be mould into any shape when mixed with water in appropriate proportions, and the long-term strength is independent of time once it is
hardened and dried. The initial setting time for the gypsum used in the study was in the order of 30 minutes when mixed with water in the ratio of 2:1. The basic physico-mechanical properties of plaster (after a curing period of two weeks in the oven at a constant temperature of $40^\circ$ C) were determined by performing several tests on 50 mm diameter specimen. The cured plaster showed a consistent uniaxial compressive strength ($\sigma_c$) of about 20 MPa, tensile strength ($\sigma_t$) of about 6.0 MPa and a Young's modulus ($E$) of 7.3 GPa. With the ability to alter strength properties by varying the mixing ratio, gypsum is a suitable material to simulate the behaviour of a number of jointed or weak soft rocks, such as coal, friable limestone, clay shale and mudstone, based on the ratios of $\sigma_c/\sigma_t$ and $\sigma_c/E$ relevant in similitude analysis (Indraratna, 1990).

6.2.2 Preparation of Bolted Joints

Fully mated joints of plaster moulds with saw tooth asperity angle of $18.5^\circ$ were cast (Figure 6.1) using a mixing ratio of 2:1 by weight of plaster and water. Although these regular shaped asperities may not ideally represent the complex, irregular and wavy shaped joint profiles observed in the field, nevertheless, they provide a simplified basis for evaluating the overall shear strength of bolted joints. The bottom block (250x75x100 mm) was cast inside the bottom mould containing the required surface profile and left for two hours to cure. The top specimen (250x75x150 mm) was then cast on top of the bottom specimen and subsequently, the whole assembly was left for an additional two hours to allow the top section to cure. Masking tape was used between two moulds to obtain a fully mated joint surface. During specimen
preparation, mild vibration was applied to the mould externally to eliminate any entrapped air inside the sample. The mould was then stripped and allowed to cure in an oven for 14 days at a constant temperature of $40^\circ C$. All the samples were then allowed to climatise to the room temperature prior to bolt installation. Figure 6.2 shows a typical joint sample.
Following sample curing, a 5 mm hole was drilled through the middle of the joint blocks, and a 3 mm \( \phi \) threaded steel bolt was installed in the hole using a Fosroc-Chemfix grout (organic resin) commonly used in jointed and stratified roofs of coal mines in Australia. For the purpose of simplicity, the bolts were installed perpendicular to the joint plane. This is a common practice in underground coal mines, where bolts are usually installed perpendicular to the horizontally bedded mine roofs. A uniform setting time, up to three hours, was allowed for all specimens before testing. The properties of the hardened resin after three hours were, \( \sigma_c = 76.5 \) MPa, \( \sigma_t = 13.5 \) MPa and \( E = 11.7 \) GPa. The change of properties of the grout beyond three hours was not significant. The uniaxial compressive strength of the model
material was found to be in the range of 18 to 22 MPa, which is close to the strength of soft roof rocks found in several underground mine openings.

6.2.3 CNS Direct Shear Testing Apparatus

The detailed description of the CNS shear apparatus has already been discussed in Chapter 3 (see Section 3.4). The cured specimens were placed inside the top and bottom shear boxes and screwed tightly to hold in position. Figure 6.3 shows one of the samples placed inside the bottom shear box. The bottom shear box was then placed inside the lower specimen holder of CNS machine. The top shear box was then inserted within the upper specimen holder. Once the shear box containing the bolted (or non-bolted) joint sample was in place, the top shear box was attached tightly to the top plate. For bolted joint, however, both shear boxes were screwed outside and then placed into the CNS specimen holder together. A 10 mm gap was kept between the top and bottom section of shear box to enable shearing of the bolted joint plane.

Figure 6.3: A typical joint sample inside the bottom shear box.
Prior to the commencement of shearing, a predetermined normal load was applied through a hydraulic jack, operated either manually or by an electric pump. The digital strain meter fitted to the normal load cell indicated the applied level of initial normal load. A LVDT mounted on top of the specimen was used to monitor the vertical displacement of the upper block during shearing.

The shear load was applied to the bottom shear box via a transverse hydraulic jack, which was connected to a strain-controlled unit. The applied shear load was recorded via digital strain meters fitted to a calibrated load cell. The rate of horizontal displacement for the shear boxes could be varied between 0.35 and 1.70 mm/min using an attached gear mechanism (helical worn transmission system). A sufficiently low and constant strain rate of 0.5 kN/mm was applied via the bottom shear box, causing it to move in a horizontal direction, while the movement of the upper shear box was restricted to the vertical direction only. The slow rate of shearing ensured fully drained behaviour of saturated infill (described in the following chapter), and allowed to compare the contribution of bolts in both unfilled and infilled joints. The dilation and the shear displacement of the joint were recorded by two LVDT’s, one mounted on top of the top shear box and the other attached to the side of the bottom shear box.

6.2.4 Laboratory Experiments

The shear behaviour of unfilled bolted joint was studied in the laboratory by conducting a test program on a series of modelled saw-tooth bolted and non-bolted
joints under Constant Normal Stiffness condition. The laboratory experiments were carried out in two steps. Firstly, a series of tests were conducted on non-bolted joints at initial normal stress levels ranging between 0.13 MPa and 3.25 MPa, respectively. Subsequently, another series of bolted joints were tested at the same initial normal stress condition. The purpose of these two series of tests was to determine the effect of bolt on the shear strength of joint, and also to determine the strength envelope of bolted and non-bolted joints. The applied normal stress was limited to a maximum of 3.25 MPa due to the capacity of the CNS equipment. A constant normal stiffness of 8.5 kN/mm was simulated via an assembly of four springs mounted on top of the top shear box as shown in Figure 3.8. A constant strain rate of 0.5 mm /min was maintained for all the shear tests conducted. At the end of each test, the specimen was brought back to its initial position by reversing the shear direction. The shear boxes were dismantled afterwards, and the mode of failure was observed. The final joint profile was mapped where warranted.

### 6.2.5 Processing of Test Data

The test results were processed based on the assumption that the normal and shear load acted uniformly on the whole joint plane. The values of the shear and normal stresses were calculated using the following equations:
6.2 \[ \tau_h = \frac{S_h}{WxL} \]  
6.3 \[ \sigma_{nh} = \frac{N_h}{WxL} \]

Where,

- \( \tau_h \) = shear stress at a shear displacement of \( h \),
- \( \sigma_{nh} \) = normal stress at a shear displacement of \( h \),
- \( S_h \) = shear load at a shear displacement of \( h \),
- \( N_h \) = normal load at a shear displacement of \( h \),
- \( W \) = width of specimen, and
- \( L \) = length of the specimen.

6.3 SHEAR BEHAVIOUR OF NON-BOLTED JOINTS

6.3.1 Shear Stress

The shear stress profiles for non-bolted joints are plotted in Figure 6.4. The peak shear strength for joint varies from 1.70 to 4.12 MPa depending on the level of applied initial normal stress, \( \sigma_{no} \). As the value of \( \sigma_{no} \) was increased, the peak of shear stress also increased. The gap between the stress profiles was significant at low \( \sigma_{no} \) levels, but gradually became marginal as \( \sigma_{no} \) was increased, indicating the shearing of asperities at high normal stress levels. The shear displacement corresponding to peak
shear stress decreased with the increasing $\sigma_{no}$ values. This is indicated by the inclined arrow as shown in Figure 6.4. The post peak behaviour of stress path was not analysed in the present study.

![Figure 6.4: Shear stress profile for non-bolted joints.](image)

**Figure 6.4: Shear stress profile for non-bolted joints.**

### 6.3.2 Dilation

Figure 6.5 shows the dilation behaviour obtained from the laboratory experiments for non-bolted joints. The initial negative dilation may be attributed to the sample compaction, closure of pores and the initial settlement of fine irregularities along the joint plane. The maximum dilation for non-bolted joints ranged between 3.07 mm and 1.07 mm, corresponding to an initial normal stress of 0.13 MPa and 3.25 MPa, respectively. As the normal stress increased, the dilation curves become ‘dome’ shaped suggesting the increased shearing of asperities at a shear displacement close to the asperity length. The dilation curves followed a downward trend after attaining the peak shear strength values.
6.3.3 Normal Stress

The normal stress profiles for the non-bolted joints are plotted in Figure 6.6. As with the dilation, the normal stress also showed a small initial drop and then began to increase steadily up to a shear displacement, which was slightly higher than the value corresponding to the peak shear stress. This steady increase in normal stress shows the main difference between the Constant Normal Stiffness testing and the conventional Constant Normal Load testing, where in the later, the stress profiles remain unchanged (flat straight lines) from the applied initial normal stress levels. The maximum values of normal stress recorded were 1.47 MPa and 3.94 MPa corresponding to the initial normal stress levels of 0.13 MPa and 3.25 MPa, respectively. The slope of the normal stress profiles decreased with increasing initial normal stress levels, suggesting more extensive shearing of asperities at elevated normal stress levels.
6.4 SHEAR BEHAVIOUR OF BOLTED JOINTS

6.4.1 Shear Stress

Figure 6.7 shows the shear stress profile for bolted joints. As expected, the peak shear stress increased with the increase in the level of applied initial normal stress, $\sigma_{no}$. The peak shear strength for bolted joint varied from 2.04 to 4.28 MPa depending on the magnitude of applied normal stress. The shear displacement at peak shear stress decreased with the increasing $\sigma_{no}$. The higher peaks attained by bolted joints are clearly attributed to the effect of bolting, as the bolted joint offered stiffer resistance (i.e. increased modulus) as compared to non-bolted joint. The shear displacement corresponding to the peak shear stress decreased with the increasing
$\sigma_{no}$, however, the rate of decrease was slightly diminished when compared with non-bolted joints.

![Figure 6.7: Shear stress profile for bolted joints.](image)

### 6.4.2 Dilation

The dilation profiles for bolted joint are plotted in Figure 6.8. Similar to non-bolted joints, the dilation profiles were characterised by initial negative values, which began to increase subsequently reaching maximum values between 2.77 mm and 0.60 mm, corresponding to an initial normal stress level of 0.13 MPa and 3.25 MPa, respectively. The dilation profiles for bolted joints appeared to be relatively flat as compared to the non-bolted joints, which could be attributed to the additional resistance offered by the bolt. Also, the peak dilation for bolted joints was smaller as compared to the non bolted joints for all applied normal stress levels. However, the difference between the dilations of bolted and non-bolted joints decreased with
increasing normal stress indicating the reduced bolt contribution at high normal stress levels. The dilation profiles of bolted joints were relatively flatter as compared to the non-bolted joints, indicating an increased degree of shearing of asperities in bolted joints due to increased joint stiffness.

Figure 6.8: Dilation profile for bolted joints.

6.4.3 Normal Stress

Figure 6.9 shows the normal stress profiles for bolted joint tests. The peak normal stress varied between 1.41 MPa and 3.68 MPa corresponding to the initial normal stress level of 0.13 MPa and 3.25 MPa, respectively. Figure 6.10 shows the percentage increase in normal stress with respect to the applied initial normal stress, \( \sigma_{no} \). It can be inferred that the increase in normal stress drops asymptotically with increasing \( \sigma_{no} \). The gap between the plots for bolted and non-bolted joints diminishes with increasing \( \sigma_{no} \), implying a reduced bolt contribution at higher normal stress.
levels. In general, the normal stress profiles for bolted joints were lower than those of non-bolted joints because of the resistance offered by the bolt in reducing or inhibiting joint dilation.

Figure 6.9: Normal stress profile for bolted joints.

Figure 6.10: Variation of percentage increase in normal stress with initial normal stress.
6.5 STRENGTH ENVELOPE

Figure 6.11 shows the strength envelopes for both bolted and non-bolted joints, which can be considered bi-linear with increasing normal stress. The gap between the two envelopes decreased marginally with the increasing $\sigma_{no}$, confirming the reduced bolt contribution and increased asperity shearing at higher $\sigma_{no}$. This is also evident from Figure 6.12 where the difference between the peak shear stresses for bolted and non-bolted joints are plotted against the respective $\sigma_{no}$. The slope of the failure envelope can be assumed to be bi-linear, implying that the starting point of asperity degradation initiates at a normal stress of around 1.25 MPa.

![Figure 6.11: Strength envelope for bolted and non-bolted joints.](image)
Figure 6.12: Variation of difference between peak shear stresses for bolted and non-bolted joint with initial normal stress.

6.6 CONCLUSIONS

A series of tests were conducted using the CNS apparatus for both bolted and non-bolted joints at an initial normal stress ($\sigma_{no}$) ranging from 0.13 MPa to 3.25 MPa. While the peak shear stress increased with the increasing level of $\sigma_{no}$, the shear displacement at peak shear stress decreased with the increasing $\sigma_{no}$ for both bolted and non-bolted joints. The bolt affected an increased peak shear stress and enhanced stiffness of the bolt-joint composite. The level of bolt contribution appeared to decrease with the increasing $\sigma_{no}$. 
The strength envelopes of both bolted and non-bolted joints showed an approximate bi-linear trend. The degradation of asperities was initiated at a normal stress of around 1.25 MPa. The dilation behaviour indicated the compression of fine irregularities at early stages and a smoothening effect at increased values of $\sigma_{no}$. The magnitudes of dilation and normal stress profiles were less for bolted joints in comparison with the non-bolted joints.

The shear tests described here revealed the importance of bolt contribution in stabilising rock joints under Constant Normal Stiffness conditions. The joint blocks tested were fully mated and no infill was introduced between them. In reality, the joints may contain weak or soft clayey infill between the joint planes, which considerably reduce the overall shear strength of the joint. The role of infill on the shear behaviour of reinforced joints still needs to be quantified. Therefore, the following chapter will focus on the shear behaviour of infilled bolted joints.
Chapter 7

SHEAR BEHAVIOUR OF INFILLED BOLTED JOINTS

7.1 INTRODUCTION

This chapter deals with the shear behaviour of infilled bolted joints under Constant Normal Stiffness conditions. The laboratory investigation was carried out using specially cast saw-toothed blocks containing 0 to 7.5 mm thick infill material and bolted with 3 mm diameter rods. The tests were conducted in a CNS testing machine in a similar manner to the testing of unfilled bolted joint. The results of the test were critically analysed and further supplemented with a mathematical model to predict a complete stress profile of the reinforced jointed blocks containing infill material, which is described in the following chapter.

7.2 LABORATORY INVESTIGATION

7.2.1 Sample Preparation

Figure 7.1 shows a typical saw toothed joint prepared from gypsum, with a layer of clay infill. The procedure for sample casting was the same as that of unfilled joints described in Section 6.2.2.
Clayey materials have been widely used in the past for laboratory studies, because clay infilled joints often contribute to significant rock mass instability (displacements). The ordinary bricklaying clay was used in the current study as it is widely available and also its composition can be varied depending on the testing requirement. Placement moisture content of about 27% was ensured for all the tests by keeping the clay material inside a sealed container. The corresponding shear strength envelope of clay is shown in Figure 7.2. While placing the clay on top of the
bottom specimen, a saw tooth shape metal strip was used to obtain the uniform thickness of the infill material all along the surface of the specimen. Figure 7.3 shows the process of placing a clay seam on the model joint. The thickness of the clay layer varied between 1.5 mm and 7.5 mm depending on the purpose of testing.

Figure 7.2: Shear strength envelope of clay infill.

Figure 7.3: Process of placing infill on top of joint surface.
Following clay layer placement, the top block was mounted on the bottom section, thus sandwiching the clay layer in between. The two blocks were then bolted together by installing a 3 mm bolt perpendicular to the joint plane using *Fosroc-Chemfix* resin as used with unfilled joints. Figure 7.4 shows the jointed specimen along with the threaded bolt (3 mm diameter) ready to be installed in the specimen. The properties of the hardened resin and the joint material have been described in Section 6.2.2.

![Figure 7.4: Installation of bolt perpendicular to the infilled joint.](image)

### 7.2.2 Testing Procedure

More than forty model joints were tested, and the laboratory experiments were carried out in two steps. Firstly, the shear tests were conducted on non-bolted joints and then similar tests were carried out with bolted joints. This testing arrangement allowed determining the effect of bolting on the shear strength of both unfilled and
infilled joints. The tests were conducted at various initial normal stress levels ranging between 0.13 MPa and 3.25 MPa, with thickness of the infill clay increased incrementally up to a maximum of 7.5 mm in five steps, providing an infill thickness to asperity height ratio (t/a) of 0, 0.3, 0.5, 1 and 1.5, respectively. In this way, the role of infill material on the overall shear strength magnitude of both bolted and non-bolted joints could be quantified. A constant normal stiffness of 8.5 kN/mm was applied via an assembly of four springs on top of the top shear box. This simulated stiffness was found to be representative of the coal measure rocks in the Illawarra region of New South Wales. An appropriate strain rate of 0.5 mm/min was maintained for all shear tests, which would also ensure a uniform drained condition of infilled joints. A sufficient gap (less than 10 mm) was allowed between the upper and lower boxes to enable unconstrained shearing of the infilled joint. A pair of thin metal plates was used along the sides of the joint plane to prevent any loss of infill material during testing.

Following testing, the collected data was processed in the same manner as described in Section 6.2.5. At the end of each test, the specimen was brought back to its initial position by reversing the shear direction. The shear boxes were dismantled afterwards, and the mode of failure was observed. The final joint profile was mapped, where warranted. The asperity profile for each samples were recorded for further investigation. Figure 7.5 shows one of the tested samples for both bolted and non-bolted joints, respectively.
7.3 SHEAR BEHAVIOUR OF INFILLED NON-BOLTED JOINTS

7.3.1 Shear Stress

The shear stress profiles for infilled non-bolted joints, with 1.5 mm (t/a=0.3) infill and 0.13 MPa initial normal stress, are plotted in Figure 7.6. The peak shear strength for joints varied between 0.14 MPa (corresponding to 7.5 mm infill and $\sigma_{no}=0.13$...
MPa) and 3.43 MPa (corresponding to no infill and $\sigma_{no}=3.25$ MPa, not shown in figure), respectively, depending on the level of applied initial normal stress, $\sigma_{no}$ and the infill thickness, $t$. The increase in the peak shear stress was affected by $\sigma_{no}$. The gap between the stress profiles was generally high at low $\sigma_{no}$, and gradually becomes marginal with the increasing $\sigma_{no}$, implying the shearing of asperities at high normal stress levels. The variation of shear stress between no infill and an infill of 1.5 mm thickness was very prominent, which was indicated by the large gap between them as shown in Figure 7.6b. Thus, a thin layer of infill was sufficient to reduce the shear strength of joints significantly.

![Graph showing variation of shear stress for infilled non-bolted joint](image)

Figure 7.6: Variation of shear stress for infilled non-bolted joint, a) according to initial normal stress, and b) according to infill thickness.
Chapter 7: Shear Behaviour of Infilled Bolted Joints

7.3.2 Normal Stress

The normal stress profiles for infilled non-bolted joints are plotted in Figure 7.7. The normal stress profiles show a small initial drop, and then the steady increase up to a shear displacement of slightly higher than at which the peak shear stress occurs. This steady increase in normal stress was attributed to the impact of the Constant Normal Stiffness testing, which is discussed in Section 6.3.3. The maximum values of the normal stress recorded were 0.08 MPa (corresponding to 7.5 mm infill and $\sigma_{no}=0.13$ MPa) and 3.04 MPa (corresponding to no infill and $\sigma_{no}=3.25$ MPa, not shown in figure), respectively, depending on $\sigma_{no}$ and $t$. The increase in normal stress values was relatively high at low infill thickness in comparison with thicker infill. In fact, there were, at times, some small decrease in normal stress was observed, when the infill thickness was around 7.5 mm as shown in Figure 7.7b. Also, the slope of the normal stress profiles decreased with the increasing $\sigma_{no}$, suggesting extensive shearing of asperities at higher normal stress levels.
1.5 mm infill thickness

Figure 7.7: Variation of normal stress for infilled non-bolted joint, a) according to initial normal stress, and b) according to infill thickness.

7.3.3 Dilation

Figure 7.8 shows the dilation behaviour obtained from the laboratory experiments for the infilled non-bolted joints. The initial negative dilation may be attributed possibly to the compaction of the infill, the closures of pores and the initial settlement of fine irregularities along the joint plane. The maximum dilation for non-bolted joints...
ranged between 3.07 mm (corresponding to no infill and $\sigma_{no}=3.25$ MPa, not shown in figure) and -0.44 mm (corresponding to 7.5 mm infill and $\sigma_{no}=0.13$ MPa), respectively, depending on $\sigma_{no}$ and t. The gap between the dilation curves was relatively higher at high infill thickness levels (e.g. between 5 mm and 7.5 mm infill) in comparison with low infill thickness levels (e.g. between 1.5 mm and 2.5 mm infill). Thus, the impact on joint dilation is highest when the t/a ratio approaches unity, and was considered as critical t/a ratio by previous researchers as discussed in Chapter 5. The dilation curves followed a downward trend after attaining the peak shear strength values.

7.3.4 Stress Path

In order to obtain the stress paths, the shear stress values were plotted against changing normal stress values. Figure 7.9 shows the stress path for infilled non-bolted joints at $\sigma_{no}=0.13$ MPa. The interesting point to note here is, the reversal of the direction of stress paths with infill thicknesses between 5 and 7.5 mm (t/a ratio of 1 and 1.5, respectively). At the stated infill thickness levels, the shearing of asperities was almost absent, and the shear behaviour was mainly governed by the infill alone. The bandwidth (i.e. the extent of variations of shear stress with respect to the normal stress at any particular t) of the stress paths tends to decrease with increasing level of infill thickness, indicating the reduced dilation or even compression of joints at those t/a ratios. This clearly demonstrates the impact of diminishing contacts between asperities with the increasing t/a ratios.
Chapter 7: Shear Behaviour of Infilled Bolted Joints

Figure 7.8: Variation of dilation for infilled non-bolted joint, a) according to initial normal stress, and b) according to infill thickness.
Figure 7.9: Stress paths for infilled non-bolted joint at an initial normal stress of 0.13 MPa.

7.4 SHEAR BEHAVIOUR OF INFILLED BOLTED JOINTS

7.4.1 Shear Stress

Figure 7.10 shows the shear stress profile for infilled bolted joints. As expected, the peak shear stress increased with the increasing \( \sigma_{no} \). The peak shear strength for bolted joint varies between 0.20 MPa (corresponding to 7.5 mm infill and \( \sigma_{no}=0.13 \) MPa) and 3.57 MPa (corresponding to no infill and \( \sigma_{no}=3.25 \) MPa, not shown in figure), respectively, depending on \( \sigma_{no} \) and t. In general, the shear displacement at peak shear stress decreased with increasing \( \sigma_{no} \). The higher peaks for bolted joints could be
attributed to the effect of bolting, as the bolted joints offered stiffer resistance (i.e. increased modulus) as compared to the non-bolted joint, which appeared to reduce because of the increasing $\sigma_{\text{no}}$. The variation of shear stress with the infill thickness (Figure 7.10b) indicated that, at low infill thickness levels, the fall in shear strength was significant. However, with increased infill thickness, the rate of fall in shear strength began to decrease indicating the reduced impact of infill beyond a certain infill thickness level.

![Graph showing variation of shear stress for infilled bolted joint](image)

Figure 7.10: Variation of shear stress for infilled bolted joint, a) according to initial normal stress, and b) according to infill thickness.
7.4.2 Normal Stress

Figure 7.11 shows the normal stress profiles for infilled bolted joints from the laboratory tests. The peak normal stress varied between 0.09 MPa (corresponding to 7.5 mm infill and $\sigma_{no}=0.13$ MPa) and 2.87 MPa (corresponding to no infill and $\sigma_{no}=3.25$ MPa, not shown in figure). Normally, the normal stress was found to increase, up to a peak level, with increasing shear displacement. However, there was one exception, in which the normal stress was found to decrease when both $\sigma_{no}$ and $t$ was high. Such behaviour might have been caused by excessive compaction of thicker infills, instead of the usual joint dilation, as observed in other situations.

7.4.3 Dilation

The dilation profiles for infilled bolted joints are plotted in Figure 7.12. Similar to non-bolted joints, the dilation profiles were marked with the initial negative values, and then began to increase, reaching the maximum values between 2.97 mm (corresponding to no infill and $\sigma_{no}=3.25$ MPa, not shown in figure) and –0.27 mm (corresponding to 7.5 mm infill and $\sigma_{no}=0.13$ MPa), respectively, depending on $\sigma_{no}$ and $t$. The dilation profiles for the bolted joints appears to be relatively flatter as compared to the non-bolted joint, which could be attributed to the additional resistance offered by a bolt. Also, the absolute value of peak dilation for the bolted joints was less as compared to the non-bolted joints for all levels of $\sigma_{no}$, except when the dilation is negative. Thus, the bolt acts to prevent either dilation or compression.
due to its axial stiffness. The critical t/a ratio appeared to be similar as of non-bolted joints as shown in Figure 7.12.

Figure 7.11: Variation of normal stress for infilled bolted joint, a) according to initial normal stress, and b) according to infill thickness.
Chapter 7: Shear Behaviour of Infilled Bolted Joints

1.5 mm infill thickness

Shear displacement (mm)

0.13 MPa initial normal stress

b) 1

0.13 MPa
0.63 MPa
1.25 MPa
1.87 MPa

Shear displacement (mm)

7.4.4 Stress Path

The stress paths for the infilled bolted joints, at the initial normal stress of 0.13 MPa, are plotted in Figure 7.13. The bandwidth of individual stress paths appeared to be narrow as compared to the non-bolted joints, which was discussed in Section 7.3.4. Clearly, the presence of the bolt across the joint plane offered a significant resistance against shearing and dilation as compared to the non-bolted joints.

Figure 7.12: Variation of dilation for infilled bolted joint, a) according to initial normal stress, and b) according to infill thickness.
Figure 7.13: Stress paths for infilled bolted joint at an initial normal stress of 0.13 MPa.

7.5 COMPARISON OF THE SHEAR BEHAVIOUR OF INFILLED BOLTED AND NON-BOLTED JOINTS

7.5.1 Overall Shear Behaviour

Figure 7.14 shows the shear stress profile of both the bolted and non-bolted joints for the various levels of initial normal stress (σn0) applications at 2.5 mm infill thickness. The bolted joints show increased stiffness and greater peak shear stress at all levels of initial normal stress. At low initial normal stress, the percentage increase in the peak shear stress is significant (about 30% at σn=0.63 MPa) in comparison with that at high normal stress (about 12% at σn=1.87 MPa). This implies that the effect of the
bolt on increasing the shear resistance is marginally reduced as the normal stress increased. The increased stiffness of the bolted joint is clearly attributed to the added stiffness of the grouted bolt.

Figure 7.14: Shear stress variation with shear displacement at 2.5 mm infill, for bolted and non-bolted joints.

Figure 7.15 illustrates non-linear strength envelopes plotted for both bolted and non-bolted joints. It is clear that at low normal stress levels (say less than 0.63 MPa), the apparent friction angle, expressed as the ratio of peak shear stress to normal stress, has increased significantly due to the presence of the bolt. Also, the apparent cohesion appears to be increased slightly. At higher normal stress ($\sigma_n > 1.25$ MPa), the apparent friction angle for the bolted and non-bolted joints is similar (i.e. the slope of two curves in Figure 7.15), indicating the diminished influence of the bolt at high normal stress levels. Figure 7.16 shows the corresponding dilation profiles of both bolted and non-bolted joints for various levels of normal stress. As expected, the dilation of the bolted joint is less than that of the non-bolted joint, thus verifying the
favourable effect of grouted bolts. The slope of the dilation curves for bolted joints is smaller in comparison with non-bolted joints, suggesting a greater degree of asperity degradation probably occurring in bolted joints. This aspect is elaborated later in this chapter.

Figure 7.15: Shear strength envelopes of bolted and non-bolted joints at 2.5 mm infill.

Figure 7.16: Dilation profiles for bolted and non-bolted joints at 2.5 mm infill.
7.5.2 Effect of Infill on Shear Strength

Figure 7.17 shows the percentage drop in peak shear strength of bolted and non-bolted joints with infill thickness, in relation to the corresponding shear strength of the unfilled non-bolted joints. A significant drop in peak shear strength was observed even with small infill thickness of 1.5 mm (t/a=0.3), for both bolted and non-bolted joints.

![Graph showing variation of % drop in peak shear strength with infill thickness.](image)

Based on the laboratory observations, the following hold for all t/a ratios:

a) Non-bolted joints show a greater drop in shear strength,

b) The greater the t/a ratio, the smaller will be the bolt contribution, and

c) The greater the initial normal stress, the smaller is the % drop in shear strength.
Moreover, the bolt contribution seems to decrease with the increasing normal stress for all \( t/a \) ratios (i.e. for \( \sigma_{no}=1.87 \) MPa, the gap between the dotted and solid line is smaller than the corresponding gap at \( \sigma_{no}=0.13 \) MPa).

Figure 7.18 illustrates the shear stress profile of bolted and non-bolted joints at an initial normal stress of 0.63 MPa. The difference between shear stress profiles for no infill and 1.5 mm thick infill (\( t/a=0.3 \)) is large for both bolted and non-bolted joints. The corresponding difference of curves between \( t=1.5 \) and \( t=2.5 \) is considerably smaller. This demonstrates that a small infill thickness is sufficient to reduce the shear strength of joint considerably, irrespective of whether the joint is bolted or not.

It is observed that the difference in peak shear strength between bolted and non-bolted joints tapers off as the infill thickness is increased from 0 to 7.5 mm. On the basis of the data plotted in Figures 7.17 and 7.18, it is clear that the contribution of the bolt in terms of increasing the joint shear strength diminishes quickly with the increasing infill thickness.

![Figure 7.18: Variation of shear stress with shear displacement for bolted and non-bolted joints at \( \sigma_{no} = 0.63 \) MPa.](image-url)
7.5.3 Profile of Shear Plane

Figure 7.19 show the profiles of shear planes for selected bolted joints at an initial normal stress of 1.25 MPa. The shear planes were estimated from the measured dilation corresponding to the horizontal displacement, and they are shown by dashed lines marked on the figure. The shear plane passes through both infill and asperity when the t/a ratio is 0 to 0.5, and slightly below the crown of the asperity when the t/a ratio is unity (Figure 7.19c). For t/a ratio greater than unity (e.g. t/a=1.5), the shear plane passes entirely through the infill (Figure 7.19d), in which case the shear behaviour is governed by the infill alone. Therefore, for infill joints tested in this study, a t/a ratio of 1 to 1.5 can be considered to be critical.

Figure 7.19: Relative location of shear plane through infilled joints at $\sigma_{no} = 1.25$ MPa.
The values (1.9, 3.8, 4.8 and 6.7) shown on the right hand side of each diagram in Figure 7.19 indicate the elevation of the shear plane for bolted joints measured in mm. The values shown within brackets (2.2, 4.0, 5.0, and 6.8) correspond to the elevation of the shear plane for non-bolted joints, at the same t/a ratios. For all t/a ratios, the shear plane for bolted joints always passes along a slightly lower elevation as compared to non-bolted joints, indicating either the reduced dilation or increased compression attributed to bolting. The maximum dilation or compression of the joint can be determined by subtracting the infill thickness from the elevation of the shear plane. Table 7.1 summarises the maximum dilation or compression for each t/a ratio, where the dilation is marked as positive and compression is marked as negative.

Table 7.1: Dilation or compression of joints for various t/a ratios

<table>
<thead>
<tr>
<th>Joint type</th>
<th>Infill thickness (mm)</th>
<th>t/a</th>
<th>Elevation of shear plane (mm)</th>
<th>Dilation or compression (mm)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-bolted</td>
<td>0</td>
<td>0</td>
<td>2.2</td>
<td>2.2 - 0 = 2.2</td>
<td>Bolt decreases dilation @14%.</td>
</tr>
<tr>
<td>Bolted</td>
<td>0</td>
<td>0</td>
<td>1.9</td>
<td>1.9 - 0 = 1.9</td>
<td></td>
</tr>
<tr>
<td>Non-bolted</td>
<td>2.5</td>
<td>0.5</td>
<td>4.0</td>
<td>4.0 - 2.5 = 1.5</td>
<td>Bolt decreases dilation @13%.</td>
</tr>
<tr>
<td>Bolted</td>
<td>2.5</td>
<td>0.5</td>
<td>3.8</td>
<td>3.8 - 2.5 = 1.3</td>
<td></td>
</tr>
<tr>
<td>Non-bolted</td>
<td>5.0</td>
<td>1.0</td>
<td>5.0</td>
<td>5.0 - 5.0 = 0</td>
<td>Slight compression of bolted joint.</td>
</tr>
<tr>
<td>Bolted</td>
<td>5.0</td>
<td>1.0</td>
<td>4.8</td>
<td>4.8 - 5.0 = -0.2</td>
<td></td>
</tr>
<tr>
<td>Non-bolted</td>
<td>7.5</td>
<td>1.5</td>
<td>6.8</td>
<td>6.8 - 7.5 = -0.7</td>
<td>Bolted joint undergoes greater compression.</td>
</tr>
<tr>
<td>Bolted</td>
<td>7.5</td>
<td>1.5</td>
<td>6.7</td>
<td>6.7 - 7.5 = -0.8</td>
<td></td>
</tr>
</tbody>
</table>
From Table 7.1, it is clear that as the infill thickness is increased, the joints undergo compression rather than dilation. This indicates that the influence of asperities decreases as t/a ratio increases. When t/a<1, the infilled joints show dilation, whereas for t/a>1, the same undergo compression. Even at high t/a ratios, the bolt demonstrates the effect of preventing dilation (increasing compression). Therefore, it can be inferred that, despite the reduced bolt effectiveness at increased t/a ratios with respect to shear strength (Figure 7.17), nevertheless the bolt maintains the ability to reduce dilation and/or increase compression of infilled joints.

7.5.4 Strength Envelopes

Figures 7.20 and 7.21 show the variation of the shear stress with the normal stress along with the corresponding strength envelopes for bolted and non-bolted joints, for 2.5 mm and 7.5 mm infill thickness respectively. At low infill thickness (2.5 mm or t/a ratio = 0.5), there exist a clear difference in strength envelopes between bolted and non-bolted joints and, as expected, the apparent friction angle or shearing resistance (i.e. shear stress to normal stress ratio) is greater for bolted joints as shown in Figure 7.20. As the infill thickness was increased to 7.5 mm or t/a=1.5 (Figure 7.21), the difference between strength envelopes of bolted and non-bolted joints appears to have reduced, verifying the earlier findings that, at high t/a ratios, the shear behaviour is governed by the infill, with little or no contribution from the bolts. At low infill thickness, the linearised CNL strength envelope (shown by dashed lines) lies significantly above the CNS envelope (Figure 7.20) and, as the infill thickness is increased (t/a>1.0), the CNL and CNS strength envelopes for both bolted and non-
bolted joints tend to become close to each other as shown in Figure 7.21. This suggests that the concept of Constant Normal Stiffness is more relevant for rough joints with little or no infill, whereas for clayey material (soils in general), the shear behaviour can be represented sufficiently by the conventional shear strength envelope (Constant Normal Load) alone.

Figure 7.20: Stress paths and strength envelopes for 2.5 mm infill thickness (t/a=0.5) for bolted and non-bolted joints.

Figure 7.21: Stress paths and strength envelopes for 7.5 mm infill thickness (t/a=1.5) for bolted and non-bolted joints.
7.6 CONCLUSIONS

It can be inferred from this study that, there occurs a significant drop in peak shear strength with relatively thin infill thickness of 1.5 mm ($t/a = 0.3$), for both bolted and non-bolted joints. The bolt contributed to the increased peak shear stress and the stiffness of the bolt-joint composite. The overall shear strength of bolted joint was increased up to 30% at low normal stress levels, but was less than 12% at high normal stress levels, thus indicating the reduced bolt effectiveness at high normal stress levels. The CNS strength envelope of infilled joints was found to be non-linear, and in bolted joints the apparent friction angle was found to increase considerably at low normal stress. However, the effect of bolting was reduced with the increased normal stress. The increase in joint shear strength due to bolting was also influenced by the infill thickness. The greater the $t/a$ ratio, the smaller the role of bolting is in influencing the joint shear strength. Nevertheless, the presence of bolt contributed to minimising the joint dilation. On the basis of the current study, bolts reduce dilation of joints when $t/a < 1$, and increase joint compression for $t/a > 1$.

Shear plane passes through the infill or asperity or both, depending on the infill thickness to asperity height ratio, $t/a$. Beyond a critical $t/a$ ratio of 1.0, the effect of bolting or the influence of asperities becomes marginal, and the joint behaviour is dictated mainly by the infill characteristics. At low infill thickness, the shear behaviour of bolted joints under CNS condition is different from its behaviour under conventional CNL condition. At $t/a$ ratios exceeding unity, the difference between the CNS and CNL envelopes is insignificant, implying that the conventional shear
strength envelope (CNL) can be adequate for representing the behaviour clayey infill. Nevertheless, for rough joints with little or no infill, the CNS behaviour is more realistic in practice, especially for underground mining conditions in bedded or jointed rock.
Chapter 8

MODELLING OF THE SHEAR BEHAVIOUR OF REINFORCED JOINTS

8.1 INTRODUCTION

In Chapter 5, various models for predicting the shear strength of both bolted and non-bolted joints were reviewed. Most of these models, based on experimental, analytical or numerical techniques have been developed to function under CNL conditions, typically applicable to shearing across planar joints. These models may not necessarily provide adequate information when applied to non-planar joints as discussed in Chapter 2. Accordingly, in this chapter the formulation of an analytical model based on the Fourier analysis has been described, along with the development of a numerical model using Universal Distinct Element Code (UDEC) to predict the shear behaviour of such joints under CNS conditions.
8.2 ANALYTICAL MODEL FOR SHEAR BEHAVIOUR OF UNFILLED REINFORCED JOINTS

8.2.1 Definition of Fourier Series and its Applicability in Joint Modelling

A function \( f(x) \) is said to be periodic with a period \( 2T \), if for all \( x \), \( f(x + 2T) = f(x) \), where \( T \) is a positive constant (see Figure 8.1). For example, \( \sin (x + 2\pi) \) is a periodic function with a period of \( 2\pi \). The Fourier expansion corresponding to \( f(x) \) can be defined as (Spigel, 1974):

\[
f(x) = \frac{a_0}{2} + \sum_{n=1}^{\infty} \left( a_n \cos \frac{n\pi x}{T} + b_n \sin \frac{n\pi x}{T} \right)
\]

(8.1)

where, the Fourier coefficients \( a_n \) and \( b_n \) are given by:

\[
a_n = \frac{1}{T} \int_{-T}^{T} f(x) \cos \frac{n\pi x}{T} \, dx
\]

(8.2)

\[
b_n = \frac{1}{T} \int_{-T}^{T} f(x) \sin \frac{n\pi x}{T} \, dx
\]

(8.3)
Fourier series have been used as a good mathematical tool for defining various continuous functions of any kind. The application of Fourier series in the field of rock mechanics has lately been introduced by Indraratna et al. (1999). When applied to rock joints, Fourier series can be used to model the joint dilation profile of both unfilled and infilled reinforced joint with reasonable accuracy. Accordingly:

\[
\delta_v(h) = \frac{a_0}{2} + \sum_{n=1}^{\infty} \left( a_n \cos \frac{2n\pi x}{T} + b_n \sin \frac{2n\pi x}{T} \right)
\]  

(8.4)

If the total shear displacement is divided into \( m \) equal parts as shown in Figure 8.2, then the Fourier coefficients \( a_n \) and \( b_n \) can be rewritten as:
Chapter 8: Modelling of the Shear Behaviour of Reinforced Joints

\[ a_n \approx \frac{2}{m} \sum_{k=0}^{m-1} y_k \cos \frac{2k\pi}{m} n \]  \hspace{1cm} (8.5)

\[ b_n \approx \frac{2}{m} \sum_{k=0}^{m-1} y_k \sin \frac{2k\pi}{m} n \]  \hspace{1cm} (8.6)

where,

\( \delta_v(h) = \) dilation at any shear displacement of \( h \),

\( a_n, b_n = \) Fourier coefficients, and

\( T = \) period of the function \( \delta_v(h) \).

Figure 8.2: Dilation profile segmented into \( m \) equal parts.
8.2.2 Prediction of Variation in Normal Stress

Once the dilation of the bolted joint is calculated for a particular shear displacement, the effective normal stress on the joint plane could be calculated using the following equation:

\[
\sigma_n(h) = \sigma_{no} + \frac{k_n \cdot \delta_v(h)}{A}
\]  

(8.7)

where,

\(\sigma_n(h)\) = effective normal stress at any shear displacement \(h\),

\(\sigma_{no}\) = initial normal stress,

\(A\) = area of joint plane; and

\[
k_n = \frac{1}{\frac{1}{k_j} + \frac{1}{k_b}}
\]  

(8.8)

where,

\(k_n\) = bolt-joint normal stiffness,

\(k_j\) = joint normal stiffness, and

\(k_b\) = bolt normal stiffness.
8.2.3 Prediction of Shear Stress

As indicated by Johnstone and Lam (1989), the joint asperity angle does not remain constant over the entire length of shear displacement, and a similar phenomenon can also be observed in the case of bolted joints. As the bolted joint is sheared, the asperities start to wear depending on the degree of applied initial normal stress on the joint plane, shear displacement and the characteristics of bolt/grout interface. Thus the shear stress of a bolt-joint composite, $\tau_h$, at any point could be given by:

$$\tau_h = \tau_{bs}(h) + \tau_{ns}(h)$$

(8.9)

If the bolt is installed at an angle $\theta$ with respect to the joint plane as shown in Figure 8.3, the shear stress component due to bolt shear stiffness is given by:

$$\tau_{bs}(h) = \frac{k_s h}{A_b / \sin \theta}$$

(8.10)

where,

$\tau_{bs}(h) = \text{shear stress component due to bolt shear stiffness at a shear displacement of } h,$

$k_s = \text{bolt shear stiffness,}$

$A_b = \text{cross sectional area of the bolt, and}$
\( \theta \) = angle of inclination of bolt with respect to the joint plane.

Figure 8.3: Orientation of bolt with respect to joint plane.

For calculating the shear stress component due to normal stiffness of the bolt-joint composite, the shear strength model (based on the energy principle) proposed by Sidel and Haberfield (1995) for non-bolted joint was used as the initial basis of the current model. In a simple form, their model can be written as:

\[
\tau(h) = \sigma_n(h) \left( \frac{\tan(\phi_b) + \tan(i)}{1 - \tan(\phi_b) \tan(i_h)} \right)
\]

(8.11)

where,

\( \tau(h) \) = shear stress of joint at a shear displacement of \( h \),

\( \sigma_n(h) \) = effective normal stress on the joint plane at a shear displacement of \( h \),

\( \phi_b \) = angle of internal friction of joint material,

\( i \) = angle of asperity at zero shear displacement, and
\( \gamma_h = \) asperity angle at an shear displacement of \( h \).

By introducing the bolt component into the above joint model, the overall shear strength of a bolt-joint composite can be represented by:

\[
\tau(h) = \frac{k_s \cdot h}{A_s / \sin \theta} + \sigma_{bn}(h) \left[ \frac{\tan(\phi_b) + \tan(i)}{1 - \tan(\phi_b) \tan(i_h)} \right]
\]  

(8.12)

where,

\( \tau(h) = \) Shear stress of bolt-joint composite at a shear displacement of \( h \),

and

\( \sigma_{bn}(h) = \) Effective normal stress on the joint plane of bolt-joint composite at a shear displacement of \( h \) and can be calculated by Equation 8.7.

The peak shear stress, \( \tau_p \), could be obtained by differentiating Equation 8.12 and then equalling it to zero. The simple solution for the above equation could not be achieved due to the complexity involved in handling the Fourier terms using ordinary calculus. The mathematical package, MATHEMATICA was subsequently used to obtain the final solution. Appendix I provides the detailed steps involved in solving the above equation. In a simplified form, the peak shear strength of a bolted joint, \( \tau_p \), is given by:

\[
\tau_p = \frac{k_s \cdot h_{\tau_p}}{A_s / \sin \theta} + \left[ \sigma_{no} + \frac{k_n}{A} \left( \frac{2\pi h_{\tau_p}}{T} \right) \right] \left[ \frac{\tan(\phi_b) + \tan(i)}{1 - \tan(\phi_b) \tan(i_{h_p})} \right]
\]
8.3 ANALYTICAL MODEL FOR SHEAR BEHAVIOUR OF INFILLED REINFORCED JOINTS

The development of analytical model for predicting the shear strength of infilled bolted joints was based on the model developed for unfilled joint and the shear strength drop. The model consisted of two parts: first, the shear strength of bolted joints without any infill was calculated (using the model as per Equation 8.13), and then the drop in shear strength due to infill was calculated using a hyperbolic model. In the final step, the shear strength of infilled bolted joint was calculated by deducting the strength drop due to infill from the unfilled joint strength.

8.3.1 Modelling of Drop in Shear Strength

8.3.1.1 Impact of infill thickness on peak shear stress

Figure 8.4 shows the peak shear stress values for bolted joints, obtained from laboratory testing and plotted against the respective t/a ratios. Clearly, there is a sharp drop in strength values from no fill to infill thickness of 1.5 mm (i.e. t/a=0.3). This
drop, however, tapers off asymptotically when the infill thickness is increased beyond 1.5 mm (t/a=0.3). The drop in shear strength is significantly influenced by the t/a ratio while its value is less than unity. As soon as the value of t/a ratio exceeds unity, the shear strength curves become almost parallel to the x-axis.

![Variation of peak shear stress with t/a ratio.](image)

**Figure 8.4: Variation of peak shear stress with t/a ratio.**

### 8.3.1.2 Relationship between normalised strength drop and t/a ratio

Figure 8.5 shows the Normalized Strength Drop (NSD), defined as the ratio of drop in peak shear strength divided by the initial normal stress, against t/a ratio for bolted joints. In order to quantify the impact of infill on the NSD of bolted joints, the shear strength of the bolted joint with no infill was taken as the reference point for calculating NSD. As can be seen from Figure 8.5, the NSD increases sharply with infill thickness as low as 1.5 mm (t/a = 0.3), indicating a sudden drop in shear strength. Further drop in NSD becomes insignificant at t/a ratios exceeding unity or
at the critical t/a value as indicated in Figure 8.5. At low normal stress values, the NSD is more sensitive to the changing t/a ratio. As the normal stress increases (say 1.25 MPa), NSD curves become almost parallel to the x-axis indicating reduced impact of normal stress on the overall drop in shear strength.

8.3.1.3 Modelling of normalised strength drop

As indicated by Duncan & Chang (1970), Indraratna et al. (1999) and others, NSD is usually modelled using the hyperbolic relationship given by Equation 8.14:

\[
NSD = \frac{t/a}{\beta + \alpha(t/a)} \quad (8.14)
\]

or,

\[
\frac{t/a}{NSD} = \beta + \alpha(t/a) \quad (8.15)
\]
where,

\[ \alpha, \beta = \text{Constants, values being dependent on the effective normal stress on the joint plane and the surface roughness of joint plane.} \]

Equation 8.15 is a straight line representing the x-axis as \( t/a \) ratios and the y-axis as \( (t/a)/\text{NSD} \) respectively. Figures 8.6a and 8.6b show the above linear relationship for the results obtained from laboratory experiments for both bolted and non-bolted joints. The slope and the intercept of these lines are also tabulated in Figure 8.6. The shear strength of bolted and non-bolted joints at no infill was taken as the respective datum levels. The correlation coefficients for the above equation for the laboratory data set were found to vary between 0.990 and 0.999 for both bolted and non-bolted joints. Therefore, Equation 8.14 can be assumed to predict NSD very accurately for the experimental data for both bolted and non-bolted joints. Thus, using Equation 8.14, the strength drop due to infill can be calculated for both type of joint with reasonable accuracy.
Chapter 8: Modelling of the Shear Behaviour of Reinforced Joints

Figure 8.6: Straight line formulation of NSD; a) for non-bolted joint, and b) for bolted joint.
8.3.2 Modelling of Shear Strength of Infilled Bolted Joints

Once the strength drop is calculated using the method described above, the shear strength of infilled bolted joint can be calculated by:

\[
\left( \tau_p \right)_{\text{infilled bolted}} = \left( \tau_p \right)_{\text{unfilled bolted}} - \Delta \tau_p \tag{8.16}
\]

where,

\[\tau_p = \text{peak shear strength of bolted joint, and}\]

\[\Delta \tau_p = \text{drop in shear strength due to infill.}\]

Now, putting the value of peak shear strength of unfilled bolted joint from Equation 8.13 in the above equation, we get:

\[
\left( \tau_p \right)_{\text{infilled bolted}} = \frac{k_s h_{\tau_p}}{A_s \sin \theta} + \left\{ \sigma_{no} + \frac{k_n}{A} \left( \frac{a_0}{2} + a_1 \cos \frac{2\pi h_{\tau_p}}{T} \right) \right\} - \sigma_{no} \left[ \frac{t/a}{\alpha - t/a + \beta} \right] \tag{8.17}
\]

where,

\[\left( \tau_p \right)_{\text{infilled bolted}} = \text{peak shear strength of infilled bolted joint.}\]
Equations 8.13 and 8.17 were used in a computer program to calculate the shear stress values at various shear displacements and at different initial normal stress conditions for unfilled and infilled bolted joints, respectively.

8.4 COMPUTER PROGRAM FOR CALCULATING FOURIER COEFFICIENTS AND THE SHEAR STRENGTH

For faster processing, an in-house computer program was written, in *Quick Basic*, to calculate the Fourier coefficients and the corresponding shear strength of reinforced joints. The computer code for this program is given in Appendix II. The input data for the program include various physical parameters relating to joint such as the joint basic friction angle, shear displacement intervals, stiffness of the system, initial normal stress etc. The outputs from the program include the Fourier coefficients, peak shear stress, and the values of shear stress, normal stress and dilation at various shear displacements. Using the output, the complete stress profiles for joints can be plotted and compared with the laboratory results. A typical sample input and output from the program is also shown in Appendix II.

8.5 VERIFICATION OF ANALYTICAL MODEL

Using the above mention computer program, the Fourier coefficients were calculated for various values of infill thickness and at different initial normal stress levels. Figure 8.7 shows the shear stress profile calculated from the model (as per Equation
8.12) and those obtained from the laboratory experiments for unfilled joints. As can be seen from Figure 8.7, the shear stress profile predicted by the Equation 8.12 was in close agreement with the laboratory results. The predictions for non-bolted joints (using the model of Indraratna et al., 1999) were closer as compared to bolted joints (using the current model presented in this thesis). This is probably due to the factor that, the shear stiffness of the bolt was assumed to be constant even at the plastic deformation stage in the model, which simplifies the actual deformation behaviour of the bolt. Figure 8.8 shows the dilation behaviour obtained from laboratory experiments and the model predictions (Equation 8.4), both for bolted and non-bolted joints without any infill. As can be seen, the dilation predictions were very close to the actual values obtained from the laboratory experiments.

Figure 8.7: Comparison of shear stress profiles predicted by the model and from laboratory tests for unfilled joints.
Figure 8.9 shows the peak shear stress of infilled bolted joints at all infill thickness levels obtained from the laboratory results and model predictions. It can be seen from the figure that by utilising the strength drop criterion, the predicted shear stress values were in close agreement with the laboratory results. The normal stress and dilation predicted from the model for 1.5 mm infill thickness along with respective laboratory results are shown in Figures 8.10 and 8.11, respectively. As expected, the model predicted the dilations values with greater accuracy as compared to shear and normal stress values. Given the recognised constraints or limitations, the model was able to predict, with reasonable accuracy, the stress and dilation parameters defined in the above Equations 8.12, 8.13 and 8.17.

![Dilation Profiles](image)

Figure 8.8: Comparison of dilation profiles predicted by the model and from laboratory tests for unfilled joints.
Figure 8.9: Comparison of peak shear stress predicted by the model and from laboratory tests for infilled bolted joints.

Figure 8.10: Comparison of normal stress profiles predicted by the model and from laboratory tests for infilled bolted joints (t=1.5 mm).
8.6 NUMERICAL MODEL OF REINFORCED JOINTS UNDER CNS CONDITIONS

Shear behaviour of reinforced joints have been studied in the past using numerical tools by various researchers (Dar and Smelser, 1990; Weishen et al., 1990; Ferrero, 1995; Marence and Swoboda, 1995; Benmokerane et al., 1996; Pellet and Egger, 1996; Kinashi et al., 1997; Windsor, 1997; and Kharchafi et al., 1998). The main advantage of numerical methods is that the model can be built or modified easily and quickly without conducting further laboratory experiments. So far, most models were developed to suit the conventional Constant Normal Load conditions, and therefore, not suitable to be used in the present study. Accordingly, a new numerical model was developed to take into consideration for the study of shear tests conducted under
Chapter 8: Modelling of the Shear Behaviour of Reinforced Joints

Constant Normal Stiffness conditions. The new model uses UDEC version 3.1 to study the shear behaviour of reinforced joints. The following write-up describes the details of this numerical model.

8.6.1 General Background

Ground reinforcement consists of bolts installed in holes drilled in the rock mass. Two types of reinforcement models are provided in UDEC: local and global reinforcement. A local reinforcement model considers only the local effect of reinforcement, where it passes through existing discontinuities. A global reinforcement model, on the other hand, considers the presence of the reinforcement along its entire length throughout the rock mass. The global reinforcement model was used in the present model formulation.

8.6.2 Global Reinforcement

In assessing the support provided by the rock reinforcement, it is often necessary to consider not only the local restraint provided by reinforcement, where it crosses discontinuities, but also the restraint to intact rock that may experience inelastic deformation in the failed region surrounding an excavation. Such situations arise in modelling inelastic deformations associated with failed rock and/or reinforcement system (e.g. bolts) in which the bonding (grout) may fail in shear over a given length of reinforcement.
Bolt elements in UDEC allow the modelling of shearing resistance along their length, as represented by the frictional resistance (bond) between the grout and the bolt surface or the host medium. The bolt is assumed to be divided into a number of segments of length, $L$, defined by nodal points located at each end. The mass of each segment is lumped at the nodal points. Figure 8.12 shows a conceptual model for representing the mechanical behaviour of a fully grouted bolt. Shearing resistance of such a bolt is represented by spring/slider connections between the structural nodes and the block zones in which nodes are located.

Figure 8.12: Conceptual model representing the mechanical behaviour of fully grouted bolt.

The behaviour of grouted bolts

The axial behaviour of conventional reinforcement systems may be assumed to be governed entirely by the reinforcing element itself. Being slender, the reinforcing
element offers little bending resistance, and is treated as one-dimensional member (constant strain elements) subjected to uniaxial tension or compression. A one-dimensional constitutive model would, thus be adequate for describing the axial behaviour of the reinforcing element. The axial stiffness is described in terms of: (a) the reinforcement cross-sectional area, $A$, and (b) the Young’s modulus, $E$. The incremental axial force, $\Delta F^t$, is thus calculated from the incremental axial displacement by:

$$\Delta F^t = -\frac{EA}{L} \Delta u^t$$ \hspace{1cm} (8.18)

where,

$$\Delta u^t = \Delta u_{1t}$$

$$= \Delta u_{1t1} + \Delta u_{2t2}$$

$$= \left(u_{1}^{[b]} - u_{1}^{[a]} \right)t_1 + \left(u_{2}^{[b]} - u_{2}^{[a]} \right)t_2$$

$u_{i}^{[a]}$, $u_{i}^{[b]}$ etc. are the displacements at the bolt nodes associated with each bolt element. Subscript 1 corresponds to the x-direction and subscript 2 to the y-direction. The subscripts [a], [b] refer to the nodes. The direction cosines $t_1$ and $t_2$ refer to the tangential (axial) direction of the bolt segment.

Figure 8.13 shows a line diagram of the axial behaviour of a grouted bolt. A tensile yield limit and a compressive yield force limit can be assigned to the bolt.
Accordingly, bolt forces cannot develop that are greater than the tensile or compressive strength limits.

![Figure 8.13: Line diagram showing the axial behaviour of bolt in UDEC.](image)

Calculation of the relative displacement at the grout/rock interface employs an interpolation scheme to compute the nodal displacement along the bolt axial direction. In UDEC, each node on the bolt is assumed to exist within constant strain triangular zone (called *host zone*) as shown in Figure 8.14. The interpolation scheme uses weighting factors, which are based on the distance to each grid points of the *host zone* as given by:

\[
\Delta u_{sp} = W_1 \Delta u_{x_1} + W_2 \Delta u_{x_2} + W_3 \Delta u_{x_3}
\] (8.19)
where,

\[ \Delta u_{xp} = \text{incremental x-component of displacement at the nodal point}, \]
\[ \Delta u_{xi} = \text{incremental grid point displacements (i = 1..3), and} \]
\[ W_i = \text{weighting factors (i = 1..3)}. \]

Figure 8.14: A typical bolt element passing through triangular block.

Figure 8.15 represents the shear behaviour of the grout annulus. During relative displacement between the bolt/grout interface and the grout/rock interface, the shear behaviour is governed by the grout shear stiffness. The maximum shear force that can be developed, per length of element, \( F_x^{\text{max}}/L \), is limited by the cohesive strength of the grout.
8.6.3 Material properties

A number of bolts were tested in the laboratory to determine the area, density, Young’s modulus, and yield force resistance of the bolt to be input into the model. However, the properties related to the grout were found to be difficult to estimate, and the grout annulus was thus assumed to behave as an elastic-perfectly plastic solid. The incremental shear force, $\Delta F^i$, caused by the incremental relative shear displacement, $\Delta u^i$, between the bolt surface and the borehole surface was related to the grout stiffness, $K_{\text{bond}}$, by the following relationship.

$$\Delta F^i = K_{\text{bond}} \Delta u^i \quad (8.20)$$
The value of $K_{\text{bond}}$ could be determined from pullout tests. Alternatively, the stiffness can be calculated from a numerical estimate for the elastic shear stress, $\tau_G$, obtained from an equation describing the shear stress at the grout/rock interface (St. John and Van Dillen, 1983):

$$\tau_G = \frac{G}{(D/2 + t) \ln(1 + 2t/D)}$$  \hspace{1cm} (8.21)

where,

- $G$ = grout shear modulus,
- $D$ = reinforcing diameter, and
- $T$ = grout annulus thickness.

Consequently, the grout shear stiffness, $K_{\text{bond}}$, is thus given by:

$$K_{\text{bond}} = \frac{2\pi G}{\ln(1 + 2t/D)}$$  \hspace{1cm} (8.22)

In UDEC, the following expression has been found to provide a reasonable estimate of $K_{\text{bond}}$ (St. John and Van Dillen, 1983), which was has been incorporated in the current formulation by the expression:

$$K_{\text{bond}} \approx \frac{2\pi G}{10 \ln(1 + 2t/D)}$$  \hspace{1cm} (8.23)
The maximum shear force per bolt length in the grout is thus the bond cohesive strength, and its value can be estimated from the results of pullout tests at various confining pressures, which at times becomes difficult to conduct in the laboratory. Otherwise, it may be approximated from the peak shear strength relationship as shown below (St. John and Van Dillen 1983):

\[ \tau_{\text{peak}} = \tau_I Q_B \]  

(8.24)

where,

\[ \tau_I = \text{approximately half of the uniaxial compressive strength of the weaker of the rock and the grout, and} \]

\[ Q_B = \text{quality of the bond between the grout and rock,} \]

\[ = 1 \text{ for perfect bonding.} \]

Neglecting the frictional confinement effects, \( S_{\text{bond}} \), may then be obtained from:

\[ S_{\text{bond}} = \pi(D+2t) \tau_{\text{peak}} \]  

(8.25)

Failure of reinforcing system does not always occur at the grout/rock interface. Failure may occur at the bolt/grout interface, which is often the case for ground reinforcement. In such cases, the shear stress should be evaluated at this interface by replacing expressions \( D+2t \) with \( D \) in Equation 8.25.
8.6.4 The Continuous Yielding model

The continuous yielding joint model (Cundall and Hart, 1984) was proposed to simulate, in a simpler manner, the internal mechanism of progressive damage of joints under shearing, and was used in the formulation of present model. It is more realistic than the standard Mohr-Coulomb joint as it considers the non-linear behaviour observed in physical tests such as joint degradation, normal stiffness, dependence on normal stress, and decrease in dilation angle with plastic shear displacement. In this model the constitutive relationship is defined by:

\[ \Delta \sigma_n = k_n \Delta u_n \]  

(8.26)

where,

\[ k_n = \text{normal stiffness} = \alpha_n \sigma_n^{\beta_n} \]  
in which \( \alpha_n \) and \( \beta_n \) are model constants.

For, shear loading, the model considers an irreversible non-linear behaviour from the onset of shearing. The shear stress increment is calculated by:

\[ \Delta \tau = k_s \Delta u_s \]  

(8.27)

where,

\[ k_s = \text{normal stiffness} = \alpha_s \sigma_s^{\beta_s} \]  
in which \( \alpha_s \) and \( \beta_s \) are model constants.
8.6.5 Conceptual CNS Model for Reinforced Joints

Figure 8.16 shows the conceptual model used in the UDEC analysis for simulating the joint conditions as tested in the laboratory. The block sizes and the asperity dimensions were proportional to the actual model dimensions used in the laboratory. The Boundary conditions applied to the model ensured the following laboratory conditions.

Figure 8.16: Conceptual CNS model of reinforced joints.
Chapter 8: Modelling of the Shear Behaviour of Reinforced Joints

- The bottom shear box moves only in x direction i.e. displacement in y direction is zero, whereas displacement in x direction is free.

- The top shear box moves only in y direction i.e. displacement in x direction is zero, whereas displacement in y direction is free.

- The spring block moves only in y direction. All the movements of the block were restricted to the lower part of the spring, which was governed by the upward movement of the top specimen. The upper part of the spring block was fixed with the rigid load cell assembly, which in turn was attached to the body of the CNS equipment.

8.6.6 UDEC Modelling of Reinforced Joints under CNS Condition

As with every other UDEC model, the first step was to create the first block. It was then split into three blocks representing lower joint block, upper joint block and the spring block. The splitting of lower and upper joint blocks involved the creation of the joint plane asperities, which was achieved by incorporating a FISH function called `placecrack` into the model. Once the blocks were created, they were discretised into appropriated sizes using the `gen quad` or `gen edge` function in UDEC. The material properties were then assigned to each block using UDEC `prop` and `change` functions. Finally the bolt was introduced into the model using `cable` function in UDEC and the boundary conditions and normal stress were applied. The following material properties were used in the model.
Joint material properties:

Bulk modulus: 1.4 GPa
Shear modulus: 0.79 GPa
Density: 1400 Kg/m³

Joint properties:

Joint normal stiffness: 0.45 GPa/m (correspond to 8.5 kN/mm over an area of 250mm x 75mm)
Joint shear stiffness: 0.045 GPa/m
Joint friction angle: 33.5°

Bolt/grout properties:

Bolt modulus (E): 98.6 GPa
Grout bond stiffness: 1.12x10⁷ N/m/m
Grout cohesion: 1.75x10⁵ N/m

Figure 8.17 shows the output from UDEC for discretised joint block and the bolt along with its node points. All the models were run for 300,000 iterative steps and saved in UDEC sav file format. Appendix III shows the UDEC code used to formulate the reinforced joint model under CNS condition.
Figure 8.17: Mesh generation by UDEC.

8.6.7 Validation of the Model

The numerical model, as described above, was validated by comparing its output with the laboratory results. Figure 8.18 indicates the variations of shear stress with shear displacement from both the numerical model and the laboratory tests for selected initial normal stress conditions. At low normal stress conditions (Figure 8.18a), both the laboratory results and the UDEC model predicted the shear stress values in close agreement with each other. As the initial normal stress was increased, there was an increasing gap between the UDEC predictions and the laboratory
results, as shown in Figure 8.18c. Such behaviour was attributed to the continuous yielding model used in this study, which did not allow the shearing of asperities. In practical situations where the high normal stress condition prevails, the shearing of asperities is inevitable before reaching the peak shear. Thus the peak shear stress at high normal stress conditions has two components; (a) the shear component due to normal stress, and (b) the shear stress change due to degradation of asperities. The component (b) is missing in UDEC model, and hence, the model predicted lower shear stress values at high normal stress conditions, when compared with the laboratory results.

Figure 8.19 shows the comparison of dilation profiles from the laboratory tests and UDEC predictions. Again, similar trends in behaviour were observed for dilation profiles, as in the case of shear stresses, i.e. at low normal stress conditions, predictions were very good, however, at high normal stress conditions, dilations were over-estimated by UDEC. Normal stress predictions from UDEC model are shown in Figure 8.20 along with laboratory results. Again, greater deviations were observed at higher normal stress conditions.
Figure 8.18: Comparison of shear stress profiles predicted by UDEC model and from laboratory tests.
Figure 8.19: Comparison of dilation profiles predicted by UDEC model and from laboratory tests.
Figure 8.20: Comparison of normal stress profiles predicted by UDEC model and from laboratory tests.
In addition to the CNS UDEC model as discussed above, a UDEC code was also written to predict the shear behaviour of reinforced joints under the CNL conditions, for the purpose of comparison. This was achieved by making the joint normal stiffness to be zero in the CNS model. Figures 8.21a and 8.21b show the shear stress predictions under both CNL and CNS conditions, at initial normal stress levels of 0.13 MPa and 1.87 MPa, respectively. In either case, the CNL and CNS predictions were close to each other at low shear displacements. At increased shear displacement, the difference between the CNL and CNS increases. Because only limited degradation of asperities occurs at low shear displacements, it can be inferred that, at small shear displacement, the CNL predictions are acceptable.

The peak shear stress predicted from the CNL model was smaller as compared to the CNS model, especially at low initial normal stress levels (see Figure 8.21a). At low CNS shear stresses, the joint shear response is influenced by the increased normal stress associated with the joint dilation. In the case of CNL, the joint dilation does not contribute to increase the shear strength, as the normal stress is kept constant. Therefore, the CNL shear stresses tend to be smaller when compared with the CNS predictions. At high normal stress conditions, the joint dilation is relatively small due to the degradation of asperities. Thus, the corresponding joint shear response, is governed mainly by the frictional properties of the joint. Therefore, both CNL and CNS models predict similar values at high initial normal stress conditions (see Figure
8.21b). In view of the above, it may be concluded that, at high normal stress levels, both CNL and CNS model can be applied with equal confidence or reliability.

Figure 8.21: Comparison of shear stress profiles predicted by UDEC model under CNL and CNS conditions.
8.7 CONCLUSION

Based on the Fourier analysis and the hyperbolic predictions, an analytical model was formulated to predict the stress and dilation profiles for both unfilled and infilled bolted joints under Constant Normal Stiffness condition. A BASIC program was written to use the analytical model to predict the Fourier coefficients, stress components, dilation and the peak shear strength. A numerical model of unfilled joints was also presented using UDEC version 3.1 to predict the shear behaviour under CNS conditions. A summary of conclusions drawn in this chapter includes:

- The predicted values of shear stress, normal stress, and dilation of unfilled bolted joint by the model were in close agreement with the laboratory results.
- Hyperbolic equation predicted the drop in shear strength for infilled bolted joints to a very high accuracy with a regression coefficient exceeding 0.99.
- The analytical model of infilled bolted joint using the unfilled model and the hyperbolic equation predicted the shear behaviour of such joints close to the laboratory observations.
- The UDEC model underestimated the shear strength when compared with laboratory testing, whereas it overestimated both normal stress and dilation especially at high initial normal stress conditions.
- The CNL and CNS predictions were dissimilar at low to medium normal stress conditions.
- Both CNL and CNS models predicted similar values at small shear displacements and as well as at high initial normal stress conditions.
9.1 CONCLUSIONS

Several aspects of the shear behaviour of fully grouted bolts under Constant Normal Stiffness condition were studied, both in the laboratory and in the field. The laboratory testing programme included; (i) the study of the shear behaviour and failure mechanism of the bolt/resin interface of two most popular bolt types, currently in use in Australian coal mines, (ii) the study of the shear behaviour of model joints made from gypsum plaster, with and without bolting, and (iii) the study of the shear behaviour of bolted and non-bolted joints containing clay infill. The laboratory study on the shear behaviour of the bolt/resin interface of fully grouted bolts was supported with field investigations in two local coal mines, namely West Cliff and Tower Collieries, in the Southern Coalfields of Sydney Basin, NSW, Australia. An analytical model was presented using Fourier transforms, to predict the shear behaviour of both unfilled and infilled bolted joints. The following paragraphs describe the main conclusions drawn from this study.
Rock Bolt and Joint Research

To date, most of the rock bolt research is concerned mainly with the estimation of rock load and the load transfer mechanisms via the bolt-ground interaction. While some of the reported research work is aimed at studying the failure mechanism of the bolt/resin interface or the rock mass itself, only a limited research work is reported on the influence of bolt surface geometry on the failure mechanism of bolt/grout interface. The past research on rock joints and their stability have been reported widely, and they were mainly conducted under the Constant Normal Load condition. Only lately, there has been limited reporting on the shear behaviour of both unfilled and infilled joints under Constant Normal Stiffness condition, yet no research work has been reported on the shear behaviour of bolted joints with infill.

Shear Behaviour of Bolt/Resin Interfaces

- The shear behaviour of the bolt surface at various confining pressures directly affects the load transfer mechanism from the rock to the bolt.
- Bolts with deeper rib profiles and wider rib spacing (e.g. bolt type I) offered higher shear resistance at low confining pressures less than 6 MPa, whereas, bolts with shallower rib profile with narrower rib spacing (e.g. bolt type II) offered marginally higher shear resistance at confining pressures beyond 6 MPa. Such findings were supported by both laboratory and field investigations.
- The maximum dilation of the bolt/resin interface occurred at a shear displacement of about 60% of the bolt rib spacing.
• In general, the shear behaviour at low normal stress and the post-peak behaviour was relatively superior for type I bolts than type II bolts.

Field Investigations and Load Transfer Mechanism of Fully Grouted Bolts

Field investigations at both West Cliff and Tower Collieries revealed the following:

• The load transfer on the bolt was influenced by; a) the confining ground stress conditions, b) the strata deformation, and c) the surface profile of the bolts. The load transfer on type I bolts (50 kN at West Cliff Colliery and 39 kN at Tower Colliery) was relatively higher as compared to type II bolts (19 kN at West Cliff Colliery and 21 kN at Tower Colliery) when subjected to low shear loading. On the other hand, under high shear loading conditions, load transfer on type II bolts was marginally higher as compared to type I bolts.

• There was no significant level of load build up on the bolts, when the development face advanced beyond 50 m away from the test site. However, the influence of the approaching of longwall face was detected when the face was around 150 m away.

• There was no significant variation of the load build up on the bolts in the two gateroads, when the face distance from the test site was more than 150m.

• The influence of front abutment pressure build up on the gateroads appears at different face positions. The load build up on the bolts in the belt road occurs when the longwall face is less than 150m from the test site, whereas, the same build up on the travelling road starts when the face position is less than 60m.
• The maximum load transferred along the bolts was found to occur at the level of maximum deformation.

• The field study showed that, under the low influence of horizontal stress (both in magnitude and the direction), type I bolt offered significantly higher shear resistance, whereas under high influence of horizontal stress, type II bolt offered marginally greater shear resistance at the bolt/resin interface. Such findings were also observed in the laboratory, and they provide a useful guide for selecting appropriate bolt types for given ground stress conditions.

Shear Behaviour of Unfilled Bolted Joints

The peak shear stress was increased with the increasing level of \( \sigma_{no} \), and the shear displacement at peak shear stress decreased with the increasing \( \sigma_{no} \) for both bolted and non-bolted joints. The bolt affected an increased peak shear stress and an enhanced stiffness of the bolt-joint composite, however, the level of bolt contribution appeared to decrease with the increasing \( \sigma_{no} \). The bolt increased the joint shear strength by more than 20% at \( \sigma_{no} = 0.13 \) MPa, whereas the corresponding increase at \( \sigma_{no} = 3.25 \) was merely 6%. The strength envelopes of both bolted and non-bolted joints showed an approximate bi-linear trend. The magnitudes of dilation and normal stress profiles were less for bolted joints in comparison with the non-bolted joints.
Shear Behaviour of Infilled Bolted Joints

The peak shear strength of both bolted and non-bolted infilled joints decreases significantly with a thin infill layer of 1.5 mm (t/a = 0.3). The overall shear strength of the bolted joint was increased by up to 30% at low normal stress levels, but not greater than 12% at high normal stress levels, indicating the reduced effectiveness of the bolt at high normal stress levels. The CNS strength envelope of infilled joints was found to be non-linear, and in bolted joints, the apparent friction angle was found to increase significantly at low normal stress. The greater the t/a ratio, the less important the role of bolting is in affecting the joint shear strength. Bolts reduce dilation of joints when t/a < 1, and increase joint compression when t/a > 1.

The shear plane passes through the infill or asperity or both, depending on the infill thickness to asperity height ratio, t/a. Beyond a critical t/a ratio of 1.0, the effect of bolting and the influence of asperities become marginal, and the joint behaviour is dictated mainly by the infill characteristics. At low infill thickness, the shear behaviour of bolted joints under CNS condition is distinctly different from the behaviour under conventional CNL. At t/a ratios exceeding unity, the difference between the CNS and CNL envelopes is marginal, implying that the conventional shear strength envelope (CNL) can adequately represent the behaviour of clayey infill. Nevertheless, for rough joints with little or no infill, the CNS behaviour is more realistic in practice, especially for underground mining conditions in bedded or jointed rock.
Modelling of the Shear Behaviour of Reinforced Joints

- The values of shear stress, normal stress, and dilation of both unfilled and infilled bolted joints predicted by the analytical model were in close agreement with the laboratory results, for $\sigma_n$ range of 0.13 MPa to 3.25 MPa.
- The drop in shear strength of infilled joints, as predicted by the hyperbolic equation, was accurate with a regression coefficient exceeding 0.99.
- The UDEC model underestimated the shear strength when compared with laboratory tested values, whereas it overestimated both the normal stress and dilation, at high initial normal stress conditions (i.e. $\sigma_{no} > 1.25$ MPa).
- For $\sigma_{no} > 1.25$ MPa, both CNL and CNS models predicted identical shear stress values at small shear movements, however, at $\sigma_{no} < 1.25$ MPa, the shear stress for both CNL and CNS were different and influenced by the value of $\sigma_{no}$.

9.2 RECOMMENDATIONS FOR FUTURE RESEARCH

The study of the shear behaviour of grouted bolts under CNS should be extended to address the following, which have not been fully addressed within the scope of this study.
Chapter 9: Conclusions and Recommendations

*Bolt surface studies:*

- Bolt surface studies reported in this thesis should be advanced to include other rigid bolt configurations as well as cable bolts.

- The bolt surface study should be extended by using numerical tools such as UDEC, FLAC, ABACUS etc., which will lead to the design of new bolt surface profiles to suit various in-situ stress conditions.

- The field investigations should be extended to include the bolt performance at a wide range of in-situ stress conditions. With the use of more extensive instrumentation, such as the measurement of in-situ stress in the instrumentation site and the use of horizontal strata movement measurement techniques, a more comprehensive design scheme for grouted bolts may be accomplished.

*Infilled bolted joint studies:*

- The laboratory studies of infill bolted joint introduced in the thesis should be extended to; (a) bolts installed at various angles to the joint plane, (b) various types of infill material, (c) joints profiles with closer representation of natural joints, and (d) varying stiffness conditions, and (e) the influence of pore pressures on infilled joint shear strength.
• Further studies may be conducted to compare the applicability of CNL and CNS techniques for various surrounding environmental conditions, such as, low, medium or high normal stress (i.e. $\sigma_{no} < 0.25$ MPa, $\sigma_{no} \approx 1.0$ MPa and $\sigma_{no} > 1.25$ MPa), thin or thick infill layer (i.e. $t/a$ ratio of as low as 0.1 and as high as 5), and small or large shear displacements (i.e. $h < 1$ mm and $h > 1$ mm), as well as the influence of pore pressures.

• The Set theory may be applied by using Venn diagrams to draw a boundary between the CNL and CNS.

• Further field investigations should be conducted to involve the in-situ study of infilled bolted joint behaviour by using in-bed shear movement measurement techniques, and validate it with analytical models based on the field data.
REFERENCES


References


Appendix I

SOLUTION OF EQUATION 8.12 FOR SHEAR STRENGTH OF REINFORCED JOINTS

The original equation is given by,

\[
\tau(h) = \frac{k_s h}{A_b / \sin \theta} + \sigma_{bn}(h) \left[ \frac{\tan(\phi_b) + \tan(i)}{1 - \tan(\phi_b) \tan(i_h)} \right]
\] (A1)

Putting the values of \(\sigma_{bn}\) from Equation 8.7 we obtain,

\[
\tau(h) = \frac{k_s h}{A_b / \sin \theta} + \left( \sigma_{no} + \frac{k_n \delta_v(h)}{A} \right) \left[ \frac{\tan(\phi_b) + \tan(i)}{1 - \tan(\phi_b) \tan(i_h)} \right]
\] (A2)

Now introducing the Fourier terms for dilation as per Equation 8.4 and assuming that the dilation function is an even function, we obtain,

\[
\tau(h) = \frac{k_s h}{A_b / \sin \theta} + \left( \sigma_{no} + \frac{k_n \delta_v(h)}{A} \left\{ \frac{a_0}{2} + \sum_{n=1}^{\infty} \left[ a_n \cos \frac{2n\pi x}{T} \right] \right\} \right) \left[ \frac{\tan(\phi_b) + \tan(i)}{1 - \tan(\phi_b) \tan(i_h)} \right]
\] (A3)
knowing that,

\[
\tan(i_h) = \frac{d(\delta_v)}{dh}
\]

\[
= -\frac{2\pi}{T} \sum_{n=1}^{\infty} \left( a_n \cos \frac{2n\pi x}{T} \right)
\]

(A4)

and substituting the value of \(\tan(i_h)\) in Equation A3, then,

\[
\tau(h) = \frac{k_s h}{A_b / \sin \theta} + \left( \sigma_{no} + \frac{k_n}{A} \left( a_0 + \sum_{n=1}^{\infty} \left( a_n \cos \frac{2n\pi x}{T} \right) \right) \right)
\]

\[
\frac{\tan(\phi_b) + \tan(i)}{1 - \tan(\phi_b) \tan \left( -\frac{2\pi}{T} \sum_{n=1}^{\infty} \left( a_n \cos \frac{2n\pi x}{T} \right) \right)}
\]

(A5)

For peak shear stress, \(\frac{d\tau(h)}{dh}\) approaches to zero, and differentiating Equation A5

with respect to \(h\), we obtain,
Appendix I: Solution of Equation 8.12

\[
\frac{d\tau}{dh} = C_1 - C_2 \left[ \frac{C_3 \sin(th) + C_3 C_4 + \left( \sigma_{\text{no}} C_4 t + \frac{a_4 C_3 C_4}{2a_1} \right) \cos(th)}{\{1 + C_4 \sin(th)\}^2} \right]
\]

(A6)

Where,

\[
C_1 = \frac{k_s \cdot h}{A_b \cdot \sin \theta}
\]

\[
C_2 = \tan\left(\phi_b\right) + \tan(i)
\]

\[
t = \frac{2\pi}{T}
\]

\[
C_3 = \frac{k_n}{A_j} a_1 t
\]

\[
C_4 = a_1 t \tan\left(\phi_b\right)
\]

for \(\frac{d\tau}{dh} = 0\), in Equation A6, then,

\[
C_4^2 \sin^2(th) + (2C_1 C_4 - C_2 C_3) \sin(th) - C_2 C_5 \cos(th) + (C_1 - C_2 C_3 C_4) = 0
\]

(A7)

Where,
Appendix I: Solution of Equation 8.12

\[ C_5 = \sigma_{no} C_4 t + \frac{a_0 C_3 C_4}{2a} \]

The above equation can be rewritten in the following form,

\[
\left( C_4^4 \right) y^4 + \left( 2C_4^2 C_6 \right) y^3 + \left( C_6^2 + 2C_7 C_4^2 + C_2 C_5 \right) y^2 + \left( 2C_6 C_7 \right) y + \left( C_7^2 - C_2 C_5 \right) = 0 \quad \text{(A8)}
\]

Where,

\[ C_6 = \left( 2C_1 C_4 - C_2 C_3 \right) \]

\[ C_7 = \left( C_1 - C_2 C_3 C_4 \right) \]

\[ y = \sin(\theta) \]

The Equation A8 is a fourth order linear equation of \( y \), which was solved using MATHEMATICA.
Program code for calculating Fourier constants and stress parameters

'PROGRAM CODE FOR REINFORCED SHEAR STRENGTH MODEL

CLS
CONST PI = 3.141592
INPUT "Input complete name of data file"; infile$

DIM maxcount AS INTEGER 'maxcount= maximum number of data points
INPUT "provide maximum number of data points"; maxcount
DIM a(0 TO maxcount) AS SINGLE, b(1 TO maxcount) AS SINGLE

OPEN infile$ FOR INPUT AS #1

LINE INPUT #1, title1$
LINE INPUT #1, title2$
INPUT #1, SDStart!, SDEnd!, SDInterval!, SW, SL, Stiffness, INS,
FricAngl, Asp_angle
Div_count% = (SDEnd! - SDStart) / SDInterval!
DIM Dilation(0 TO Div_count%) AS SINGLE
DIM EstDil(0 TO Div_count%) AS SINGLE
DIM NS(0 TO Div_count%) AS SINGLE
DIM ShearStss(0 TO Div_count%) AS SINGLE
DIM Slope(0 TO Div_count%) AS SINGLE

FOR x = 0 TO Div_count%
    INPUT #1, Dilation(x)
NEXT x

INPUT #1, Infill_type%
IF Infill_type% = 0 THEN GOTO EndRead
INPUT #1, TAStart, TAEnd, NumStep%, Alfa, Beta, Rf

DIM PeakShearDrop(0 TO NumStep%) AS SINGLE
DIM InfillPeak(0 TO NumStep%) AS SINGLE

EndRead:
CLOSE #1

'Calculation of Fourier coefficients

FourierCoeff:

REDIM a(0 TO maxcount) AS SINGLE, b(1 TO maxcount) AS SINGLE

FOR n = 0 TO maxcount
    FOR k = 0 TO Div_count% - 1
Appendix IIA: Program Code for Fourier Coefficients and Shear Strength

\[
a(n) = a(n) + \text{Dilation}(k) \times \cos(2 \times \pi \times k \times n / \text{Div\_count})
\]

NEXT k
\[
a(n) = a(n) \times 2 / \text{Div\_count}
\]

NEXT n

FOR n = 1 TO maxcount
  FOR k = 0 TO Div\_count% - 1
    \[
b(n) = b(n) + \text{Dilation}(k) \times \sin(2 \times \pi \times k \times n / \text{Div\_count})
\]
    NEXT k
  b(n) = b(n) \times 2 / \text{Div\_count}
  NEXT n

'Calculation of dilation

Dilation:

EstDil(0) = 0
NS(0) = INS
T = SDEnd! - SDStart! ; k = 1

FOR x = (SDStart! + SDInterval!) TO SDEnd! STEP SDInterval!
  y = 0
  FOR n = 1 TO maxcount
    y = y + a(n) \times \cos(2 \times \pi \times n \times x / T) + b(n) \times \sin(2 \times \pi \times n \times x / T)
  NEXT n
  y = a(0) / 2 + y
  EstDil(k) = y
  NS(k) = INS + Stiffness \times EstDil(k) \times 1000 / (SW \times SL)
  k = k + 1

NEXT

'Calculation of shear stress

ShearStss:

ShearStss(0) = 0; PeakShear = 0; NormAtPeak = INS

FOR k = 1 TO Div\_count%
  Slope(k) = ATN((EstDil(k) - EstDil(k - 1)) / SDInterval!)
  ShearStss(k) = 1000 \times (1.72 + .65 \times \log(k)) / (SL \times SW) + NS(k)
    \times (\tan(FricAngl \times \pi / \text{180}) + \tan(Asp\_angle \times \pi / \text{180})) / (1 - \tan(\pi \times FricAngl / \text{180}) \times \text{TAN}(\text{Slope}(k)))
  IF PeakShear < ShearStss(k) THEN
    PeakShear = ShearStss(k)
    NormAtPeak = NS(k)
    DisAtPeak = k \times SDInterval! + SDStart!
  ELSE
    END IF
  NEXT k

261
Appendix II A: Program Code for Fourier Coefficients and Shear Strength

Calculation of impact of infill

IF Infill_type% = 0 THEN GOTO Printing
TAINvl = (TAEnd - TAsStart) / NumStep%

FOR k = 0 TO NumStep%
    PeakShearDrop(k) = INS * (TAsStart + TAINvl * k) / (Alfa * (TAsStart + TAINvl * k) + Beta)
    InfillPeak(k) = PeakShear - PeakShearDrop(k)
    MaxDrop = PeakShear - (INS * Rf / Alfa)
    IF InfillPeak(k) < MaxDrop THEN InfillPeak(k) = MaxDrop
NEXT k

INPUT "Input the value of t/a ratio for calculating the infill Peak shear stress "; TA
PeakShearDrop = INS * TA / (Alfa * TA + Beta)
InfillPeak = PeakShear - PeakShearDrop

Printing of final output to a date file

printoutput:
INPUT "Type result file name"; Outfile$
OPEN Outfile$ FOR OUTPUT AS #2

PRINT #2, "Fourier Coefficients for:"
PRINT #2, title1$: PRINT #2,
PRINT #2, title1$: PRINT #2,
PRINT #2, " a  b"
PRINT #2, USING " (0) ###.#####"; a(0)

FOR n = 1 TO maxcount
    PRINT #2, USING " (##) ; a(n); "
    PRINT #2, USING " ###.#####"; b(n)
NEXT n

PRINT #2, USING "Disp(mm) Dil(mm)  NS(MPa)  Slope(o)  SS(MPa)"

FOR k = 0 TO Div_count%
    PRINT #2, USING "####.#####"; SDStart! + k * SDInterval!;
    PRINT #2, USING "####.#####"; EstDil(k);
    PRINT #2, USING " ####.#####"; NS(k);
    PRINT #2, USING "#####.#####"; Slope(k) * 180 / Pi;
    PRINT #2, USING "#####.#####"; ShearStss(k)
NEXT k

PRINT #2, USING "Peak shear stress ####.#### MPa "; PeakShear

262
Appendix IIA: Program Code for Fourier Coefficients and Shear Strength

PRINT #2, USING "Normal stress at peak shear ####.##### MPa "; NormAtPeak
PRINT #2, USING "Shear displacement at peak shear ###.## mm "; DisAtPeak

IF Infill_type% = 0 THEN GOTO EndPrint
PRINT #2,
PRINT #2, " T/A PeakShearDrop InfillPeak 

FOR k = 0 TO NumStep%
  PRINT #2, USING "###.## "; (TAStart + TAIntvl * k);
  PRINT #2, USING " ####.##### "; PeakShearDrop(k);
  PRINT #2, USING " ####.##### "; InfillPeak(k)
NEXT k

PRINT #2, USING " PeakShearDrop = ####.#### at T/A = ##.## ";
PeakShearDrop; TA
PRINT #2, USING " InfillPeak = ####.#### at T/A = ##.## "; InfillPeak; TA

CLOSE #2

PRINT "The output from the program is written to"; Outfile$; "file"

EndPrint:

END
Appendix: IIB

Program code for converting data into input format

'PROGRAM CODE FOR CONVERTING DATA FROM EXCEL TO BASIC PROGRAM INPUT FORMAT

CLS
INPUT "Input complete name of data file"; infile$
OPEN infile$ FOR INPUT AS #1
    INPUT #1, nlim%
    DIM Dilation(0 TO nlim%) AS SINGLE

DilRead:
    FOR x = 0 TO nlim% - 1
        INPUT #1, Dilation(x)
    NEXT x

EndRead: CLOSE #1

printToFile:
INPUT "Type name of the file for output to be written"; outfile$
OPEN outfile$ FOR OUTPUT AS #2
    FOR k = 0 TO nlim% - 1
        PRINT #2, USING "###.###"; Dilation(k);
    NEXT k

    j = nlim% - 1
    PRINT #2,

    FOR k = 0 TO nlim% - 1
        PRINT #2, USING "###.###"; Dilation(j);
        j = j - 1
    NEXT k

FilePrintEnd:

PRINT "Please use the data in "; outfile$; "for input to the Fourier analysis program"
CLOSE #2

END
Appendix: IIC

Sample input file

Data file for Fourier Coefficient calculation
No of data points = 24/0.25 + 1
0 24 .25
75 250
8.5 0.63 37.5 18.5
0.00 0.03 0.07 0.10 0.15 0.21 0.26 0.32 0.39 0.46 0.53 0.58
0.67 0.73 0.81 0.88 0.96 1.05 1.11 1.18 1.23 1.30 1.35 1.39
1.46 1.53 1.59 1.67 1.74 1.81 1.88 1.91 1.96 2.01 2.06 2.11
2.16 2.21 2.26 2.30 2.35 2.39 2.43 2.46 2.49 2.53 2.56 2.58
2.61
2.58 2.56 2.53 2.49 2.46 2.43 2.39 2.35 2.30 2.26 2.21 2.16
2.11 2.06 2.01 1.96 1.91 1.88 1.81 1.74 1.67 1.59 1.53 1.46
1.39 1.35 1.30 1.23 1.18 1.11 1.05 0.96 0.88 0.81 0.73 0.67
0.58 0.53 0.46 0.39 0.32 0.26 0.21 0.15 0.10 0.07 0.03 0.00
0
Appendix: IID

Sample output file

Fourier Coefficients for:
No of data points = 24/0.25 + 1

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<th>SS(MPa)</th>
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23.750  0.07768  0.66522  -3.37  0.95096  0.95096  0.95096  0.95096
24.000  0.07275  0.66298  -1.13  0.96962  0.96962  0.96962  0.96962

Peak shear stress  2.4075 MPa
Normal stress at peak shear   1.7014 MPa
Shear displacement at peak shear  10.00 mm
Appendix: IIIA

UDEC code for calculating shear strength of reinforced joints under CNS condition

;File "C:\Itasca\UDEC\boltedjn.dat"
;
;UDEC code for determining the shear strength of reinforced joints
;
;******************************************************************************************
;
def placecrack ;define fish fn to create joint profile
;
crack_ht = 0.005
crack_ln = 0.015
;
stx = -0.06
sty = 0
fnx2 = -0.045
fny2 = -0.005
;
loop while stx < 0.3
	command
		crack (stx,sty) (fnx2,fny2)
	endcommand
	stx = fnx2
	sty = fny2
	fnx2 = fnx2 + crack_ln
	fny2 = fny2 + crack_ht
;
	command
		crack (stx,sty) (fnx2,fny2)
	endcommand
	stx = fnx2
	sty = fny2
	fnx2 = fnx2 + crack_ln
	fny2 = fny2 - crack_ht
;
endloop
end
;
;******************************************************************************************
;
;Block creation
;
round 0.001
bl (-0.06,-0.1) (-0.06,0.30) (0.30,0.30) (0.30,-0.1)
placecrack ;call fish function placecrack

;crack (0,0.30) (0,0)
crack (0.24,0.30) (0.24,0)
;
crack (0,0.15) (0.24,0.15)
;
del range (-0.06,0) (0,0.30) ;left side
del range (0.25,0.30) (0,0.30) ;right side
;
;**********************************************************
;Mesh generation
;
gen edge 0.03 range bl 4162 ;upper block
gen edge 0.03 range bl 689 ;lower block
gen quad 0.1 0.1 range bl 4502 ;spring block
;
;**********************************************************
;Assign material and cable properties
;
prop mat = 1 d = 1.4e-3 k = 1400 g = 792 ; all
change mat=2 range bl 4502 ; spring
prop mat=2 d=5e-3 k=23 g=34.5
;
;**********************************************************
;Define joint model(C-Y model)
;
change jcons=3
set jcondf=3
set ovtol=0.01
set add_dil on
set del off
;
;**********************************************************
;Define joint properties
;
prop jmat=l jkn=453 jks=45.3 jen=0.0 jes=0.0 jfric=33.5
prop jmat=1 jif=66.0 jr=5e3
change jmat=2 range 0,0.24 0.14,0.16
prop jmat=2 jkn=453 jks=10 jen=0.0 jes=0.0 jfric=0.0
prop jmat=2 jif=0.0 jr=0.0
;
;**********************************************************
;Place cable
;
cable (0.125,-0.09) (0.125,0.09) 19 3 181e-6 5
;
;properties of cable
;
prop mat=3 cb_dens 7500 cb_ymod=98.6e9
prop mat=3 cb_yield=232e3 cb_ycomp=1e10

270
Appendix IIIA: UDEC Code for Calculating the Shear Strength of Reinforced Joints

; properties of grout
;
prop mat=5 cb_kbond=1.12e7 cb_sbond=1.75e5
;
;******************************************************************************
;
; apply boundary conditions
;
bound xvel=0 range -0.01, 0.01 0.01, 0.30
bound xvel=0 range 0.23, 0.25 0.01, 0.30
bound yvel=0 range -0.06, 0.3 -0.11, -0.09
;
;******************************************************************************
;
; apply normal load
;
bound stress (0, 0, -1.25) range 0, 0.24 0.29, 0.31
;
;******************************************************************************
;
hist unbal
step 3000
bound yvel=0 range 0, 0.24 0.29, 0.31
;
call jn_str.fis ; call fish program for cal. stress values
Appendix: IIIB

**Fish program code for calculating stress values**

```plaintext
; File "C:\Itasca\UDEC\jn_str.fis"
;
; Fish function to calculate the average stress/disp.
;
; Variable initialization
;
; def av_str
whilestepping
sstav=0.0
nstav=0.0
njdisp=0.0
sjdisp=0.0
ncon=0
jl=0.100 ; joint length
;
**********************************************************

; Calculation of average shear strength and disp
;
; ic=contact_head
loop while ic # 0
;   if c_y(ic) > -0.0025 then
;     if c_y(ic) < 0.001 then
;       if c_y(ic) > -0.00295 then
;         neon = neon + 1
;         sstav = sstav + c_sforce(ic)
;         nstav = nstav + c_nforce(ic)
;         njdisp = njdisp + c_ndis(ic)
;         sjdisp = sjdisp + c_sdis(ic)
;     endif
;   endif
;   ic = c_next(ic)
endloop
if ncon # 0
  sstav = sstav/jl
  nstav = nstav/jl
  njdisp = njdisp/ncon
  sjdisp = sjdisp/ncon
endif
end
;
**********************************************************

; Recording of calculated data
;
reset hist jdisp
hist unbal nc 1
```
Appendix IIB: UDEC FISH Function used in Model Formulation

hist n=1000 sstav nstav njdisp sjdisp
hist n=1000 ydisp 0.050,0.011 ;top-middle bolt
hist n=1000 xdisp 0.050,0.005 ;left-middle bolt
hist n=1000 ydisp 0.050,0 ;middle-b/r interface

;*****************************************************
;Apply shear load by imposing shear velocity on bottom block
;
bou xvel = -0.03 range (-0.001,0.101) (0,1) ;top block
step 300000
save JS125.sav