Experimental and computational study on rock bolt modelling and its application on a new type of energy-absorbing rock bolt

Yokota, Yasuhiro

2019

Yokota, Y. (2019). Experimental and computational study on rock bolt modelling and its application on a new type of energy-absorbing rock bolt. Doctoral thesis, Nanyang Technological University, Singapore.

<https://hdl.handle.net/10356/90006>

<https://doi.org/10.32657/10220/49891>

**NANYANG
TECHNOLOGICAL
UNIVERSITY**

SINGAPORE

**EXPERIMENTAL AND COMPUTATIONAL STUDY ON
ROCK BOLT MODELLING AND ITS APPLICATION
ON A NEW TYPE OF ENERGY-ABSORBING ROCK
BOLT**

YOKOTA YASUHIRO

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

2018

**EXPERIMENTAL AND COMPUTATIONAL STUDY ON
ROCK BOLT MODELLING AND ITS APPLICATION
ON A NEW TYPE OF ENERGY-ABSORBING ROCK
BOLT**

YOKOTA YASUHIRO

School of Civil and Environmental Engineering

A thesis submitted to the Nanyang Technological University
in partial fulfilment of the requirement for the degree of
Doctor of Philosophy

Statement of Originality

I hereby certify that the work embodied in this thesis is the result of original research, is free of plagiarised materials, and has not been submitted for a higher degree to any other University or Institution.

27 July 2019

.....

Date

Yasuhiro Yokota

Yasuhiro Yokota

Supervisor Declaration Statement

I have reviewed the content and presentation style of this thesis and declare it is free of plagiarism and of sufficient grammatical clarity to be examined. To the best of my knowledge, the research and writing are those of the candidate except as acknowledged in the Author Attribution Statement. I confirm that the investigations were conducted in accord with the ethics policies and integrity standards of Nanyang Technological University and that the research data are presented honestly and without prejudice.

27 July 2019

.....
Date

.....
Zhao Zhiye

Authorship Attribution Statement

This thesis contains material from 3 paper(s) published in the following peer reviewed journals where I was the first and/or corresponding author.

Chapter 3 and a part of Chapter 4 are published as:

Yasuhiro Yokota, Zhiye Zhao, Wen Nie, Kensuke Date, Keita Iwano and Yuko Okada. Experimental and Numerical Study on the Interface Behaviour Between the Rock Bolt and Bond Material. Rock Mechanics and Rock Engineering (2018). <https://doi.org/10.1007/s00603-018-1629-4>

The contributions of the co-authors are as follows:

- Associate Prof Zhao and Mr Date provided the initial project direction and important suggestions.
- I prepared the manuscript drafts. The manuscript was revised by Associate Prof Zhao, Dr Nie and Mr Iwano.
- I designed the experiment study and also analysed the data obtained from the laboratory tests.
- Mr Iwano and Ms Okada performed all the laboratory work at Kajima Technical Research Institute in Japan.
- Dr Nie provided guidance in the simulation of the laboratory tests with DDA and created the prototype simulation model.
- I modified a simulation model and established a reliable simulation model which can reproduce laboratory tests.

A part of Chapter 3 is also published as:

J. Shang, Y. Yokota, Z. Zhao and W. Dang. DEM simulation of mortar-bolt interface behaviour subjected to shearing. *Construction and Building Materials* 185, 120-137 (2018). <https://doi.org/10.1016/j.conbuildmat.2018.07.044>

The contributions of the co-authors are as follows:

- Associate Prof Zhao provided the initial project direction and edited the manuscript drafts.
- Dr Shang prepared the manuscript drafts. The manuscript was revised by me and Dr Dang.
- I designed the experiment study and also analysed the data obtained from the laboratory tests.
- Dr Shang created DEM simulation model and did parametric studies.

Chapter 4 is published as:

Yasuhiro Yokota, Zhiye Zhao, Junlong Shang, Wen Nie, Kensuke Date, Keita Iwano and Yuko Okada. Effect of bolt configuration on the interface behaviour between a rock bolt and bond material: A comprehensive DDA investigation. *Computers and Geotechnics* 105, 116-128 (2019).

<https://doi.org/10.1016/j.compgeo.2018.09.017>

The contributions of the co-authors are as follows:

- Associate Prof Zhao and Mr Date provided the initial project direction and important suggestions.
- I prepared the manuscript drafts. The manuscript was revised by Associate Prof Zhao and Dr Shang.
- I designed the experiment study and also analysed the data obtained from the laboratory tests.
- Mr Iwano and Ms Okada performed all the laboratory work at Kajima Technical Research Institute in Japan.

- Dr Nie provided guidance in the simulation of the laboratory tests with DDA and created the prototype simulation model.
- I modified a simulation model and did all parametric studies with DDA.

27 July 2019

.....

Date

Yasuhiro Yokota

.....

Yasuhiro Yokota

Acknowledgement

This thesis is written about our research achievement which had been obtained through collaborative research between Nanyang Centre for Underground Space (NCUS) and Kajima Technical Research Institute (KaTRI) from January 2016 to December 2018.

First of all, I would like to thank Associate Professor Zhao Zhiye from School of Civil and Environmental Engineering, Nanyang Technological University, who helped me find this avenue of research to begin with, and also was of great assistance in developing this particular experiment and numerical simulation. Secondly, thanks also go to Dr Nie Wen and Dr Shang Junlong, who are my seniors in our research group, for her/his helpful suggestions about numerical simulation and friendly support regarding all of our research. Thirdly, I would like to thank Assistant Professor Wu Wei and his Ph D students, for their great suggestion and instruction on the usage of the high resolution 3D scanning.

In addition to them, I wish to thank Mr. Kensuke Date, who is my senior colleague and Chief Research engineer in Kajima Technical Research Institute Singapore, for coordinating this collaborative research work without any significant problems. In addition, I am grateful to Mr. Keita Iwano, Mr. Yu Koizumi, Mr. Takaaki Inuzuka, Ms. Yuko Okada in Kajima Technical Research Institute and also all of engineers in KFC Co. Ltd for assistance with experiments in Chapters 3 and 5. Finally, I cannot say thank you enough, my family Arisa, Taichi and Momoka. I always appreciate their great support.

Table of Contents

Chapter 1.....	17
Introduction.....	17
1.1. General background	17
1.2. Key objectives	20
1.3. Methodology.....	22
1.4. Thesis organisation	24
Chapter 2.....	25
Literature review on rock bolting system	25
2.1. Fundamental functions and reinforcement effects	25
2.1.1. Suspension effect.....	25
2.1.2. Beam composition effect	27
2.1.3. Internal pressure effect.....	27
2.1.4. Ground arch effect.....	30
2.1.5. Ground reinforcing effect	31
2.2. Types of rock bolts	32
2.2.1. Mechanically anchored rock bolts	32
2.2.2. Resin anchored rock bolts	33
2.2.3. Fully grouted rock bolts	35
2.2.4. Friction rock bolts	36
2.2.5. Injected type rock bolts	38
2.2.6. Combination bolts	39
2.2.7. Energy-absorbing rock bolts	40

2.3. Research review (experiment)	42
2.4. Research review (numerical simulation).....	59
2.5. Rock bolt under severe geological conditions	64
2.6. Summary	73
Chapter 3.....	75
Debonding process of the interface between rock bolts and bond material	
.....	75
3.1. Outline of shear test.....	76
3.1.1. Shear test equipment.....	76
3.1.2. Preparation for shear tests	82
3.1.3. Shear test procedure	82
3.2. Shear test cases	84
3.3. Shear test results	89
3.3.1. Comparison of crack distribution	89
3.3.2. Comparison of load – displacement curve.....	95
3.3.3. Further discussions.....	100
3.3.4. Summary of shear tests	103
3.4. Summary	105
3.4.1. Simplified rock bolt model without ribs	105
3.4.2. Simplified rock bolt model with ribs	105
Chapter 4.....	107
Numerical modelling using discontinuous deformation analysis.....	107
4.1. Introduction to discontinuous deformation analysis	108
4.1.1. History of discontinuous deformation analysis.....	108

4.1.2. Main characteristics of discontinuous deformation analysis	112
4.1.3. Application of discontinuous deformation analysis	114
4.2. Numerical simulation of shear tests.....	117
4.2.1. Simulation model	117
4.2.2. Simulation cases.....	120
4.2.3. Numerical Simulation results	121
4.3. Effect of rock bolt configuration.....	129
4.3.1. Numerical simulation cases and results of basic cases	129
4.3.2. Results and discussion.....	134
4.3.3. Classification of induced cracks.....	153
4.3.4. Implications of the study and recommendation for rock bolt configuration	156
4.4. Summary.....	158
Chapter 5.....	161
Development of new rock bolt with DDA.....	161
rock bolt model	161
5.1. Simulation of the detailed anchor model with DDA.....	163
5.2. Laboratory pull-out tests with prototypes.....	171
5.3. Bolt installation test.....	176
5.4. Summary.....	179
Chapter 6.....	181
Evaluation of the supporting effect of.....	181
the new DC bolt.....	181
6.1. Brief introduction of DDA rock bolt element.....	181

6.2. Simulation of pull-out test with DDA based DC element.....	183
6.3. Tunnel excavation example simulated by DDA based DC bolts	191
6.4. Summary	196
Chapter 7.....	197
Conclusion and recommendations.....	197
7.1. Conclusion.....	197
7.2. Recommendations	201
References	203

Summary

This thesis aims to study the rock bolt modelling and the rock bolt design using both laboratory tests and numerical simulations. It is well known that rock bolts are essential supports for rock tunnels and caverns. According to previous studies, the interface behaviour between the rock bolt and bond material, including the debonding process and the crack propagation, is one of the most important factors significantly affecting the reinforcement effect and support capacity of rock bolts. However, there is still a lack of comprehensive understanding on the detailed debonding process along the boundary between the rock bolt and bond material, so this research aims to fill in the gap on the lack of understanding in the interface behaviour and debonding mechanism.

In the first part of this study (Chapter 3 and 4), shear tests were carried out with a simplified rock bolt model to directly observe the interface behaviour during the shear tests. Next, the shear test results were reproduced with the discontinuous deformation analysis (DDA) which can simulate the crack distributions. As a result, it was concluded that the DDA can be an effective tool to evaluate the interface behaviour between the rock bolt and the bond material. Afterwards, the effects of bolt configurations on the bolt-mortar interface behaviour were investigated with the verified DDA model. The recommended values for the key rock bolt parameters (i.e. rib angle, rib height, rib interval, rib

shape) were obtained through the DDA simulations which take into account the crack initiation and propagation into the bond material.

In the latter part of this study (Chapters 5 and 6), a new type of energy-absorbing rock bolt was designed and its supporting mechanism was investigated with the DDA-based rock bolt model. This type of rock bolt can follow the large tunnel deformation, and it can control the final displacement by the specially designed anchor. Several laboratory pull-out tests were carried out with the prototypes of newly-developed energy-absorbing rock bolts, and the DDA-based rock bolt element which can simulate the pull-out test process was developed. With this rock bolt element, the tunnel excavation was simulated, and it was found that the new energy-absorbing rock bolt can be useful for the tunnel excavation under the squeezing conditions.

The research findings from this study enable us to better understand the detailed reinforcing mechanism and the supporting effect of rock bolts. And also, from the practical point of view, this study can contribute to design the optimum profile for the rock bolts through parametric studies using the developed DDA models. Furthermore, tunnel excavation under severe conditions such as squeezing conditions can be improved by utilising the newly-developed energy-absorbing rock bolt.

List of Publications

Published journal papers:

1. Yokota, Y., Zhao, Z. Y., Shang, J., Nie, W., Date, K., Iwano, K. and Okada, Y. (2019) “Effect of bolt configuration on the interface behaviour between a rock bolt and bond material: A comprehensive DDA investigation”, Computers and Geotechnics 105, pp. 116-128.
<https://doi.org/10.1016/j.compgeo.2018.09.017>
2. Yokota, Y., Zhao, Z. Y., Nie, W., Date, K., Iwano, K. and Okada, Y. (2018) “Experimental and numerical study on the interface behaviour between the rock bolt and bond material” Rock Mechanics and Rock Engineering. <https://doi.org/10.1007/s00603-018-1629-4>
3. Shang, J., Yokota, Y., Zhao, Z. Y., Dang, W. (2018) “DEM simulation of mortar-bolt interface behaviour subjected to shearing”, Construction and Building Materials 185, pp. 120–37.
<https://doi.org/10.1016/j.conbuildmat.2018.07.044>

Submitted journal papers:

4. **Yokota, Y.**, Zhao, Z. Y., Nie, W., Date, K., Iwano, K., Koizumi, Y. and Okada, Y. (2019) “Development of a new deformation-controlled rock bolt: numerical modelling and laboratory verification”, Tunnelling and Underground Space Technology
5. Nie, W., Zhao, Z. Y., **Yokota, Y.** and Guo, W. (2018) “Bond-slip modelling of the rockbolt element using 2D-DDA method”, Tunnelling and Underground Space Technology

International conference papers:

6. **Yokota, Y.**, Zhao, Z. Y., Nie, W., Date, K., Iwano, K., Okada, Y. and Shimizu, S. (2018) “Interface behaviour between the rock bolt & bond material”, ARMS10, Singapore (**Outstanding paper award**)
7. Zhao, Z. Y., Nie, W., **Yokota, Y.**, Xiao, F. and Bhagya, J. L. (2018) “Rock Mechanics and Rock Engineering in Singapore”, ARMS10, Singapore (**Keynote lecture**)
8. **Yokota, Y.**, Zhao, Z. Y., Nie, W., Date, K., Iwano, K. and Okada, Y. (2018) “Interface behaviour along the boundary between the rock bolt and bond material”, EUROCK2018, Vol 2, pp. 1341-1347

List of Figures

Figure 1.1 Four principal components of a reinforcement system ..	18
Figure 2.1 Dilatancy caused by roughness of the rupture surface ..	29
Figure 2.2 Ground arch zone around a circular tunnel.....	31
Figure 2.3 Components of a mechanically anchored rock bolt	33
Figure 2.4 Typical two component resin cartridge used for anchoring	34
Figure 2.5 Components of a resin anchored rock bolt with provision for grouting (Hoek, 2006)	34
Figure 2.6 Grouted dowel using a deformed bar	36
Figure 2.7 Split Set stabilizer (Hoek, 2006)	37
Figure 2.8 Outline view of swellex bolt (Hoek, 2006).....	37
Figure 2.9 Injection methods for rock bolts	39
Figure 2.10 CT bolt (Combination Tube bolt) (modified after DSI, 2017).....	40
Figure 2.11 Cone bolt.....	41
Figure 2.12 D bolt	42
Figure 2.13 Several failure patterns of the reinforced concrete	43
Figure 2.14 Several failure patterns of the reinforced concrete	44
Figure 2.15 Cone shaped cracks and longitudinal splitting cracks.	46
Figure 2.16 Stress and deformation in a grouted anchor	47
Figure 2.17 Comparison results of theoretical model with those of experiment (modified after Farmer, 1975)	47
Figure 2.18 Main cracks observed by Aydan during shear tests	50

Figure 2.19 Forces and displacements associated with a stressed cable grouted into a borehole in rock (Hoek, 2006)	53
Figure 2.20 Schematic images of fixing material during pull-out tests	54
Figure 2.21 Zig-zag surface bolt profile utilized for Bond Strength Model.....	55
Figure 2.22 Results of pull out test conducted by Stillborg	55
Figure 2.23 Observation results after pull out tests using X-ray CT scan, modified from the original journal	57
Figure 2.24 Test pieces of pull out tests.....	57
Figure 2.25 Crack initiation, propagation and coalescence of an incipient joint with DEM (Shang et al., 2018).....	61
Figure 2.26 Pull-out simulation with DDA	62
Figure 2.27 Lattice model of Brazilian tension test.....	62
Figure 2.28 Numerical simulation results of flaw inclination angle 45°	63
Figure 2.29 Support system damaged by rock burst.....	65
Figure 2.30 Sheared-off steel arch by squeezing behaviour	65
Figure 2.31 Cone bolt and supporting mechanism.....	68
Figure 2.32 Garford solid bolt and supporting mechanism	68
Figure 2.33 D bolt and supporting mechanism	69
Figure 2.34 TH profile with sliding connection.....	70
Figure 2.35 High deformable concrete (hiDCon).....	71
Figure 2.36 Lining stress controller (LSC)	72
Figure 2.37 Yielding supports in Gotthard tunnel.....	72

Figure 3.1 Overall view of the portable shear box (PHI – 10)	78
Figure 3.2 Longitudinal view of portable shear box assembly	78
Figure 3.3 Pressure maintainer on normal load application system	79
Figure 3.4 Test sample mounted in its mould	80
Figure 3.5 Test sample held with the locating clamp.....	80
Figure 3.6 Flow of preparation works.....	83
Figure 3.7 Shear test equipment.....	84
Figure 3.8 Schematic view of the shear test.....	84
Figure 3.9 Image of simplified rock bolt model (Upper right: Rib angle=30°, Lower left=60°, Lower right=90°).....	85
Figure 3.10 Estimation model of interface shear strength in boreholes (Aydan et al., 1995).....	87
Figure 3.11 The relationship between the effective confining pressure and rock deformation modulus.....	88
Figure 3.12 Photos after shear tests without ribs under a confining pressure of 2.0 MPa	89
Figure 3.13 Image manipulation during PIV analysis (White et al., 2003).....	90
Figure 3.14 Measurement conceptual image with a camera and laser pointer (Winkelbach et al., 2006)	91
Figure 3.15 Detailed observation results with Low-strength mortar (UCS = 10 MPa).....	93
Figure 3.16 Detailed observation results with High-strength mortar (UCS = 30 MPa)	94

Figure 3.17 Load – displacement curve (Laboratory tests)	96
Figure 3.18 Relationship between shear stress and normal stress. 97	
Figure 3.19 Influence of main rock bolt parameters.....	102
Figure 4.1 DEM(Cundall, 1971).....	110
Figure 4.2 DDA (Shi, 1988)	110
Figure 4.3 Classification for representative discontinuous analyses	112
Figure 4.4 Failure of a Brazilian disc with an initial hole.....	115
Figure 4.5 Simulation result of rock cavern excavating in rock mass	116
Figure 4.6 Simulation mesh by DDA.....	118
Figure 4.7 Tri-axial test results (left: UCS=30MPa, right: UCS=10MPa).....	118
Figure 4.8 Mohr-Coulomb criterion with a tension cut off in DDA	120
Figure 4.9 Load-displacement curve obtained from sensitivity analyses.....	123
Figure 4.10 DDA results after the shearing simulation (Upper) and picture of the test piece which was taken after the shear test (Lower)	123
Figure 4.11 Load – displacement curve obtained from numerical simulations(Left) and experiments (Right)	124
Figure 4.12 DDA results after shearing simulation.....	127
Figure 4.13 Load-displacement curve (numerical simulation).....	128
Figure 4.14 Simulation mesh for the basic model.....	132

Figure 4.15 Comparison of the results from the DDA simulations (left) and experimental tests (right)	133
Figure 4.16 Comparison of the load-displacement curves obtained from the DDA simulations (left) and experimental tests (right)	134
Figure 4.17 Horizontal stress and crack distribution	137
Figure 4.18 Load-displacement curve with different rib angle	137
(e) Figure 4.19 Displacement vector	138
Figure 4.20 Slip behaviour observed during laboratory tests.....	138
Figure 4.21 Relationship between rib angle and shear strength (Confining pressure = 0.5 MPa)	139
Figure 4.22 Relationship between rib angle and shear strength (Confining pressure = 2.0 MPa)	139
Figure 4.23 Horizontal stress and crack distribution	141
Figure 4.24 Load-displacement curve with different mortar thickness	142
Figure 4.25 Relationship between α and shear strength	142
Figure 4.26 Horizontal stress and crack distribution	146
Figure 4.27 Displacement vector.....	147
Figure 4.28 Load-displacement curve with different rib space	148
Figure 4.29 Relationship between rib space and shear strength ..	148
Figure 4.30 Horizontal stress and crack distribution	151
Figure 4.31 Load-displacement curve with different rib shape.....	152
Figure 4.32 Relationship between rib shape and shear strength..	152
Figure 5.1 Proposed tri-linear shaped load-displacement curve ...	162

Figure 5.2 Conceptual image of the deformation-controlled rock bolt	162
Figure 5.3 DDA model for the deformation-controlled rock bolt ...	163
Figure 5.4 Simulation results of the horizontal stress and the crack distribution	164
Figure 5.5 Load-displacement curve obtained from the DDA simulation	165
Figure 5.6 DDA simulation cases with different interface properties	167
Figure 5.7 Horizontal stress and crack distribution at the primary peak loading.....	168
Figure 5.8 Load-displacement curve obtained from the DDA simulations.....	168
Figure 5.9 Supporting mechanism of a deformation-controlled rock bolt.....	170
Figure 5.10 Prototype of deformation-controlled rock bolt	172
Figure 5.11 Photo of the laboratory pull-out test condition.....	173
Figure 5.12 Test specimen dismantled after removing the half-split pipe.....	173
Figure 5.13 Load-displacement curves obtained from laboratory pull-out tests.....	175
Figure 5.14 Acrylic pipes (left) and deformation-controlled rock bolts (right)	176
Figure 5.15 Acrylic pipes filled up with mortar (left) and bolt installation (right)	177

Figure 5.16 Mortar filling condition between the anchor and the ring (upper) and behind the anchor (lower)	178
Figure 6.1 A tri-linear bond-slip model with DDA	184
Figure 6.2 DDA simulation model of the pull-out test	185
Figure 6.3 DDA rock bolt model for the pull-out test simulation ..	186
Figure 6.4 Shear force distribution during DDA pull-out simulations	188
Figure 6.5 Axial force distribution during DDA pull-out simulations	189
Figure 6.6 Load-displacement curves obtained from DDA pull-out simulations.....	190
Figure 6.7 DDA simulation model for the tunnel excavation	192
Figure 6.8 Shear force distribution after the excavation	194
Figure 6.9 Axial force distribution after the excavation	194
Figure 6.10 Monitoring result of the rock displacement.....	195

List of Tables

Table 2.1 Functions and supporting effects of rock bolting	26
Table 2.2 Typical types of cable bolts summarized by Windsor.....	52
Table 3.1 Specifications for the portable shear box, displacement transducer and pressure transducer	81
Table 3.2 Laboratory test cases.....	86
Table 3.3 Image of dominant mortar failure patterns under 2.0MPa confining pressure	104
Table 4.1 Input data for rock bolt	119
Table 4.2 Input data for mortar model	119
Table 4.3 DDA simulation cases.....	120
Table 4.4 Summary of DDA results without rib	125
Table 4.5 Summary of DDA results with rib	128
Table 4.6 List of DDA simulation cases and the configurations....	131
Table 4.7 Input data for DDA simulation (Mortar)	132
Table 4.8 Classification for the mortar failure pattern.....	153
Table 4.9 Recommendation for the rock bolt configuration	156
Table 5.1 Input data for DDA simulations (anchor model).....	163
Table 5.2 Test cases and specifications of the rock bolt	172
Table 6.1 DDA simulation cases (pull-out test model)	186
Table 6.2 Rock bolt specifications (pull-out test model)	186
Table 6.3 Rock mass properties.....	192

Chapter 1

Introduction

1.1. General background

Rock bolts are essential supports for both mining and civil engineering during excavation of tunnels or large rock caverns and a slope stabilization process. Rock bolting system has been dramatically developed over the last four decades in the area of understanding the reinforcement mechanism of rock bolts and advancement of rock bolting system itself. Owing to various support mechanisms of rock bolting and their effects which will be described in Section 2.1, rock bolting system enables rock masses, which is mainly comprised of intact rock and joints, to enhance its capacity for strength as much as possible. In other words, rock mass could be supported by themselves with rock bolting system. As a result, a collapse of rock blocks along rock joints and large rock deformation can be restrained.

Windsor (1997) reviewed reinforcement system of rock bolting. In this review, he classified rock bolting into three types, namely rock bolts (generally less than 3m in length), cable bolts (generally between 3m and 15m) and ground anchors (typically more than 10m in length). However, according to recent tunnel construction reports, rock bolts which are between 4m and 6m in length seem to be utilised when geological conditions are poor. Furthermore, Windsor showed four principal components of a reinforcement system as illustrated in Figure 1.1.

- (0)The rock
- (1)The element (e.g. rock bolt)
- (2)The internal fixture (e.g. mortar or resin) and
- (3)The external fixture (e.g. face plate)

Figure 1.1 Four principal components of a reinforcement system

(Windsor, 1997)

The surrounding rock is not usually considered as a critical component in a reinforcing system. But rock strength and confining pressure would affect the reinforcing effect of rock bolting significantly. The internal fixture is a bond material such as a mortar grout and resin.

As for the history of rock bolting, Stillborg (1994) described that rock bolts had been utilised in the coal mining in Europe since the end of 19th century. The primary purpose of rock bolting was to prevent the unsteady rocks from falling by tying or suspending from the fresh rocks. At that time, rock bolts were called roof bolts, and they were produced by only cutting the tip of rebar and fixed to the borehole by Hammer striking. Therefore, rock bolts were sometimes fallen out from the borehole due to the vibration of explosion during the later excavation. Their

reinforcement effects were not considered very useful. After fully grouted rock bolts had been widely recognised, the reinforcement effects of rock bolting were finally understood in the civil engineering field. In addition to this, after systematically installed rock bolts as one of support for tunnel excavation, were proposed in “Water Power magazine”, rock bolting system was widely accepted all over the world (Rabcewicz 1964, 1964, 1965). Nowadays, more than 100 million rock bolts are utilised every year.

Recently, conventional tunnelling methods (or the drill & blasting tunneling methods, the new Austrian tunnelling methods), which usually employed systematically installed rock bolts, have been frequently applied for the construction of tunnels/caverns under a high overburden and with a large cross section. Under such adverse conditions, it is a concern that tunnel support materials may yield because of huge ground pressure and large tunnel deformation. As a result, a tunnel may lose its stability due to a reduced effectiveness of the tunnel supports. Therefore, rock bolt, as an essential component of tunnel supports for excavation is becoming more important.

1.2. Key objectives

According to previous studies which are summarised in Chapter 2, the interface behaviour between the rock bolt and the bond material can be considered as one of the most important factor affecting the reinforcement effect of the rock bolt. Interface behaviour includes debonding process between the rock bolt and the bond material, and also the crack initiation and propagation into the bond material.

However, laboratory tests or field tests were mainly carried out to evaluate the supporting mechanism of rock bolts in the past studies, whereby the test cases had to be limited. Besides, it is hard to observe the interface behaviour during the laboratory tests, and therefore, most observations were conducted after the experiments. In other words, there is little information available on the debonding process or crack propagation during the laboratory tests. Furthermore, there is a lack of numerical simulation models which can simulate the detailed interface behaviour between the rock bolt and the bond material.

The key objectives in this study are as follows:

- By comparing the laboratory test results with the numerical simulation results, an accurate numerical modelling which can reproduce the detailed bolt-grout interface behaviour will be developed (Chapter 3 and 4). As a result, the detailed reinforcement mechanism of rock bolts can be understood.
- Using the verified numerical models, the influence of the different rock bolt configurations on the reinforcement effect of rock bolt

will be evaluated (Chapter 4). Obtained results can be utilised for the optimum design of rock bolts.

- Using the verified numerical model, a new type of rock bolt which can withstand large tunnel displacement will be designed. By utilising the numerical model, the supporting mechanism of a new rock bolt will be investigated (Chapter 5).
- By carrying out the laboratory tests with prototypes, the characteristic curve of the new rock bolt will be obtained. In addition, the new rock bolt element will be developed and the simulation of the tunnel excavation will be carried out in order to evaluate the reinforcing effect of a newly-developed rock bolt (Chapter 6).

1.3. Methodology

In the first part of this research, both the experiment test and the numerical analysis were carried out in order to better understand the detailed interface behaviour between the rock bolt and the bond material. As for the laboratory test, a lot of shear tests were carried out, and the discontinuous deformation analysis (DDA) was used for the numerical simulations.

First of all, shear tests with simplified rock bolt models were carried out. In these experiments, the effects of the bolt configuration (with/without rock bolt ribs, rib angle), mortar strength and confining pressure on the crack propagation and load-displacement curve were evaluated. As the next step, not only load – displacement curve but also the crack propagation were simulated with DDA. Although finite element methods (FEM) or finite difference methods (FDM) are generally used for the rock bolt modelling, they cannot simulate the interface behaviour (especially on the crack propagation) and evaluate its behaviour after cracks have been generated. By utilising the DDA which is one of discontinuous analysis methods, the shear tests can be reproduced more precisely. The DDA based rock bolt models have never been done before according to the literature review.

After the verification that DDA can simulate the interface behaviour in detail during shear tests, parametric studies were conducted using the DDA rock bolt models. By doing so, the optimum rock bolt shape and mortar strength can be designed depending on the various confining pressure.

In the latter part of this research, a new type of energy-absorbing rock bolt, which can resist a large deformation, was designed and its supporting effect was verified with laboratory tests and numerical simulations.

As a first step to develop the new rock bolt which can tolerate large deformation, the anchor profile of rock bolt was determined using the DDA numerical simulation model which can reproduce the interface behaviour in detail. Then, the laboratory pull-out tests were carried out with several prototypes of new rock bolts. Finally, the DDA rock bolt element was developed by comparing the DDA pull-out simulation results with laboratory test results. And the reinforcing effect of the new rock bolt was verified by carrying out the simulation of a tunnel excavation problem.

1.4. Thesis organisation

Chapter 1 presents the introduction about the basic explanation for rock bolting and key objectives and methodology of this research. In Chapter 2, the literature review, including history and function of rock bolting and generally used models, will be provided. Chapter 3 will mainly describe laboratory tests, including the illustration of the shear test equipment, test cases, test results and discussion. Chapter 4 will represent DDA simulation which is comprised of the outline of DDA, simulation cases, simulation results and discussion. In Chapter 5, the design of a new rock bolt using DDA shear test model will be presented and the supporting mechanism of the new rock bolt will be described. Furthermore, the result of laboratory pull-out tests will be summarised. Chapter 6 will explain the development of the DDA rock bolt element, and the simulation results of a tunnel excavation example. Finally, the study will be concluded in Chapter 7.

Chapter 2

Literature review on rock bolting system

2.1. Fundamental functions and reinforcement effects

Rock bolting is considered as a simple supporting method where the rebar is installed and fixed to the ground. Therefore, its behaviour seems to be straightforward. In the past, a number of essential functions and reinforcement effects of rock bolting have been proposed all over the world.

In 1955, Rabcewicz recommended to classify its functions and supporting effects. Furthermore, some professional associations such as Japanese Society of Civil Engineering (2000) started to explain the supporting mechanism, see Table 2.1.

2.1.1. Suspension effect

Suspension effect is the simplest mechanism of rock bolting. By being suspended from fresh rocks, loosened rocks after blasting can be stabilised. It is more efficient for fractured rocks when rock bolts are utilised together with shotcrete than rock bolt only.

In northern Europe regions such as Norway and Sweden, the rock mass is typically comprised of homogeneous and fresh hard rocks, and there are fewer joints in the rock mass. In such a geological condition, the rock structure becomes stable by only controlling the joint behaviour well. Therefore, there has been a number of research results regarding the

Table 2.1 Functions and supporting effects of rock bolting
(modified after JSCE, 2000)

Effect of Rock bolting	Conceptual Sketch
i)Suspension Effect Has the effect of stabilizing on the ground, rocks which were loosened by blasting. More effective in fractured rock condition in combination with shotcrete.	
ii)Beam Composition Effect Application of rock bolt makes a number of rock strata tighten, and transmits shear stress through rock joints. By this way, it has an effect of functioning as "a composite beam".	
iii)Bearing Capacity Effect Has the effect of maintaining the durability and strength of the ground, by increasing internal pressure of tunnel ground surface by tension force of rock bolt.	
iv)Ground Arch Effect Has the effect of creating the ground arch structure by increasing the ground strength as a result of "internal Pressure Effect".	
v)Ground Reinforcing Effect Has the effect of reinforcing the physical property of the ground. Application of rock bolt improves shear strength of ground, and residual strength after yield point.	

suspension effect of rock bolting. Even in poor geological conditions (e.g., many joints, difficult to identify their strike and dip, and their behaviour), the suspension plays an important role for stabilising the rock mass. This is because only partial unstable rocks around the tunnel sometimes cause the whole rock structure to be unstable. For instance, small rock falls from a tunnel wall will prevent surrounding rocks from creating the ground arch successfully.

2.1.2. Beam composition effect

Rock bolts tie hard rock layers and soft rock layers in the alternation strata to combine them together, thereby transmitting shear stress through rock joints. By doing so, the composite beam can be formed around the tunnel. Since the ground behaviour becomes similar to the composite beam, macroscopic ground stiffness can be increased, and also ground collapse can be restrained and controlled.

2.1.3. Internal pressure effect

Internal pressure effect enables to maintain the durability and strength of rock mass by increasing internal pressure of tunnel surface with the tension force of rock bolts. This function of internal pressure is one of the most remarkable actions of rock bolting. This is the reason why an axial force measurement of rock bolt is considered as one of the most important field monitoring items during a tunnel construction. As the axial force is caused by ground deformation, ground conditions around the tunnel excavation area are expected to be subjected triaxial compressive

stress conditions. If rock bolting achieves this, the ground around the tunnel wall should be stable even when the tunnel is excavated under high-stress conditions.

However, the following issue has to be considered practically. For example, in case that each rock bolt supports the ground area of 1 m^2 and tolerates 10 tonnes of axial force, the generated confining pressure will be only 0.1 MPa. This value is a small figure compared to the initial rock stress. But rock mass is the discontinuous material which contains a number of rock joints. In such case, Goodman (1989) indicated that most of the rock mass is able to be strengthened by providing confining pressure when cracks interact with each other. As shown in Figure 2.1, the rocks can move easily along their rock joints unless they are confined. On the other hand, by increasing the confinement pressure, much higher energy should be necessary to deform along an uneven joint surface. Hence, fractured rock masses with slightly increased confinement pressure can have ten times larger strength. These are the reasons why rock bolting is useful for tunnelling in rock.

Figure 2.1 Dilatancy caused by roughness of the rupture surface
(modified after Goodman, 1989)

2.1.4. Ground arch effect

Suspension effect, beam composition effect and internal pressure effect show the function of single rock bolt. Meanwhile, the ground arch effect is meaningful only when rock bolts are installed systematically.

When a circular tunnel is excavated, the ground arch structure will be created around the tunnel wall. As shown in Figure 2.2, the ground surrounding the tunnel deform inward excavated area after the tunnel excavation in the ideal homogeneous underground (Geo fronte 2009). However, only elastic deformation can be occurred because of the tunnel shape. Moreover, deformation of rock blocks is constrained by contacting with and pushing against adjacent rock blocks. As a result, the circumference direction compressive force can be generated and propagated into the rock mass. Consequently, this behavior enables the excavated tunnel to be stabilised. Even though the tunnel is excavated in fractured rocks, this phenomenon can be seen due to same reasons as the arch bridge and arch dome. If rock bolt was installed systematically, as a result of internal pressure effect, rock bolting helps to create this ground arch structure by increasing the ground strength.

Figure 2.2 Ground arch zone around a circular tunnel
(modified after Geo fronte, 2009)

2.1.5. Ground reinforcing effect

Same as a ground arch effect, ground reinforcement effect is also the result of suspension effect, beam composition effect and internal pressure effect. The ground where rock bolts were installed behaves as structure material comprised of ground, steel and bond material such as mortar or resin. The mechanical property of steel is more than ten times larger than that of the ground. Therefore, a mechanical property of ground can be improved macroscopically.

2.2. Types of rock bolts

As described earlier, rock bolts have been widely used for almost last half a century, and many kinds of rock bolts have been developed to catch up various demands emerged in both mining and civil engineering (Hoek, 2006). Six major rock bolts will be introduced in this section.

2.2.1. Mechanically anchored rock bolts

Although an expansion shell rock bolt anchor is widely accepted, the fundamental principle is the same as another anchor typed rock bolts. As shown in Figure 2.3, the expansion shell rock bolt is mainly comprised of the tapered cone with an internal thread, a pair of wedges held in place by a bail. Expansion shell rock bolt anchors are very useful for hard rocks. On the other hand, they are not used for soft ground and fractured rocks with various rock joints because it is easy to deform and fail at the contact point with their wedge grips. In such ground conditions, resin or mortar anchored rock bolts which are described later are more suitable than mechanical anchored rock bolts.

Figure 2.3 Components of a mechanically anchored rock bolt
(Hoek, 2006)

2.2.2. Resin anchored rock bolts

Resin anchored rock bolts can be utilised for soft grounds or for excavation by blasting. In such cases, mechanically anchored rock bolts cannot produce proper reinforcing effects. A typical resin product is comprised of two component cartridges. The one is a resin, and the other is a catalyst for hardening shown in Figure 2.4. Cartridges are pushed into a borehole and broken by the rod edge. After that, resins are mixing by drilling. Generally, within several minutes, the resin becomes hard. These kinds of rock bolts can be applied to rocks including soft shale and mudstone for which expansion shell rock bolt anchors are not suitable. Figure 2.5 shows typical set-up for resin anchored rock bolts.

Figure 2.4 Typical two component resin cartridge used for anchoring and grouting rock bolts (Hoek, 2006)

Figure 2.5 Components of a resin anchored rock bolt with provision for grouting (Hoek, 2006)

As shown in Figure 2.5, several slow-setting resin cartridges are set behind the fast-setting anchor cartridges. Usually, slow-setting grout cartridges start to harden 30 minutes after the drill mixing. Meanwhile, fast-setting anchor cartridges become hardened soon, so the bolt is tensioned only a few moments after the mixing. This tension is fixed as slow-setting grout gradually become harder. Finally, rock bolt becomes a fully tensioned and fully grouted bolt. However, resin anchored rock bolts may have the long term rust problem and the reaction problem toward the groundwater, and also are expensive compared to the fully cement grouted rock bolts.

2.2.3. Fully grouted rock bolts

Tensioned rock bolts provide positive forces to the rock mass, while reinforcement effects of dowels, such as fully cement grouted rock bolts, are generated by the axial forces when ground deformation occurs. Figure 2.6 shows typical fully cement grouted rock bolts which are currently the most commonly used. For the bond materials, viscosity cement grout (typically a 0.3 to 0.35 water/cement ratio) are utilised, whereby the grout will not overflow from boreholes for rock bolts. A whole borehole can be filled with this grout using a grout installation hose. After filling up, rock bolts are installed.

Figure 2.6 Grouted dowel using a deformed bar
inserted into a grout-filled hole (Hoek, 2006)

2.2.4. Friction rock bolts

These kinds of rock bolt are fixed by the friction between the borehole wall and bolt surface. There are two types of friction rock bolts. One is “slit spring tube type (Split Set stabilisers)”, and the other one is “expansion steel tube type (Swellex bolt)”. However, the latter rock bolt is more common, especially in the civil engineering.

Split Set stabilisers were developed by Scott (1976, 1983). As shown in Figure 2.7, this bolt is comprised of the slotted high strength steel tube and faceplate. By installing it into the rather small sized borehole, the radial spring force is generated. As a result of this force, the friction force is produced along the entire rock bolt length, whereby the reinforcing effect is also created.

Figure 2.8 shows the outline picture of the swellex bolt. As shown in this figure, these kinds of rock bolts were folded when they were manufactured. After installing them into the borehole, rock bolts are

Figure 2.7 Split Set stabilizer (Hoek, 2006)

Figure 2.8 Outline view of swellex bolt (Hoek, 2006)

expanded using high pressured water (approximately 30 MPa) and contacted with the borehole wall, whereby the friction force between them can be generated. While the steel tube is expanding, the surrounding rock is expected to be consolidated. However, to obtain appropriate pulling out force, rock mass needs to be competent to some extent.

The problem regarding corrosion is one of the serious issues for friction rock bolts since their steel surfaces are contacted with rock or ground water directly. Galvanising the steel tube is the best solution currently and has been already applied. Meanwhile, friction rock bolts have a significant advantage over their installation time compared to the other typical rock bolt system. In fact, a total cost of friction rock bolts including their installation time is cheaper than that of most other rock bolt systems. Moreover, these friction rock bolts also have the other benefit. Their qualities are less affected or lowered by water inflow during installation.

2.2.5. Injected type rock bolts

Before installing the rock bolt, typically, injected type rock bolts are attached with the hole collar plug and injecting tube at the collar of the bolt and also connected to the air tube along the entire bolt. Recently, alternative methods for injection have become popular to enhance the performance of grouting. In this case, the injected tube is connected along the whole length of rock bolts, whereby bond materials are injected from the edge of the rock bolt. Bolts for injected type rock bolts are same as those for fully cemented grout rock bolts (shown in Figure 2.9).

Figure 2.9 Injection methods for rock bolts

2.2.6. Combination bolts

Combination bolts are developed to utilise for both advantages over the mechanical rock bolts and injected type rock bolts. One of the most common combination bolts is CT bolts shown in Figure 2.10 (DSI, 2017), CT stands for Combination Tube. Immediately after installing this bolt, the point anchor support is activated, and later this bolt is

fully grouted by injection. Since the surface of CT bolt is not only covered with bond materials but also coated by polypropylene sleeve, whereby they are extremely resistant to corrosion (called as a double coating).

Figure 2.10 CT bolt (Combination Tube bolt) (modified after DSI, 2017)

2.2.7. Energy-absorbing rock bolts

Recently, especially in the deep mining field, the latest rock bolts which are called as “energy-absorbing rock bolts” have been developed. These rock bolts can bear high load capacity and accommodate significant ground deformation. There are two primary energy-absorbing rock bolts, one is cone bolt (Jager, 1992, Ortlepp, 1992), and the other one is D bolt (Li, 2010).

Regarding the cone bolt, the loading force generated by the dilating rock is transferred to the tendon which is located at the edge of the rock bolt. As this force becomes large and reaches the yielding point, the cone is ploughing inside the surrounding grout materials (shown in Figure 2.11

(Li et al., 2014)). By doing so, they can absorb much energy from the surrounding rock.

Meanwhile, the D bolt is fully encapsulated in the borehole using either cement grout or resin. As shown in Figure 2.12 (Li, 2010), this bolt is mainly comprised of multiple anchor points and deformable sections with smooth surfaces. The flat deformable sections allow debonding and significant deformation. Multiple anchor points provide continued support even after several sections fail.

However, these energy-absorbing rock bolts have not yet applied to the civil engineering tunnel. The possible reasons are described in the later section.

Figure 2.11 Cone bolt

(Li et al., 2014)

Figure 2.12 D bolt

(Li, 2010)

2.3. Research review (experiment)

There have been many studies on rock bolting system to understand its effects and reinforcing mechanism.

First of all, in the nineteenth seventies, many researchers started to study the reinforcing bar in the concrete which is the pioneer in the research for rock bolting.

Lutz (1970) conducted a lot of pull-out tests of a reinforcing bar to study the bond force of smooth and deformed reinforcing bars. According to his research, the bond strength of smooth reinforcing bar depends on the chemical adhesion and friction between the bolt and concrete after slipping. However, the chemical adhesion does not influence on its reinforcement effects strongly, and the friction never generates unless the debonding between the bolt and concrete occurs.

As for the deformed reinforcing bar, he concluded that the slipping takes place in following two ways.

- 1) Rock bolt ribs split the concrete via a wedging behaviour.
- 2) Rock bolt ribs crash the concrete

When ribs crash the concrete, concrete powders store in front of ribs. Besides, when the slipping and separation occur, additional transverse cracks and splitting cracks are likely to be generated. The large deformation cannot happen unless these transverse cracks and radial cracks occur around the boundary between the bolt and concrete. Also, he found that several failure patterns of the reinforced concrete which are primary cracks (major failure mode), dense minor radial cracks and cone cracks as shown in Figure 2.13.

Figure 2.13 Several failure patterns of the reinforced concrete

(Lutz, 1970)

Goto (1971) also conducted a number of laboratory tests and studied the bond behaviour between the concrete and reinforcing bar. Especially, injecting the red ink into the test specimen after tests enabled him to observe the crack propagation into the concrete material in detail. Due to its remarkable achievements, these cracks found by using the ink are called as Goto crack. He reported that a number of internal cracks were generated around the reinforcing bar. In the case that the bolt stress was less than 100 MPa, these cone-shaped internal cracks were created from the ribs near primary cracks. Moreover, he found that these internal cracks also increased and they were generated from almost all ribs as the bolt stress increased. He also pointed out that the internal cracks leant toward the nearest primary crack as shown in Figure 2.14. The leant angle was from 45 degrees to 80 degrees, and he summarised that this angle depends on the rib shape and anchorage type.

Figure 2.14 Several failure patterns of the reinforced concrete

(modified after Goto, 1971)

Goto's research was enriched by Tepfers (1973, 1979) who had done a number of laboratory tests and presented the analytical solution for the tensile stress distribution of the concrete. According to his experiments, two kinds of cracks in the concrete were observed during pull-out tests. The first one is cone-shaped cracks, and the other one is radial splitting cracks. Both cracks are generated at the interface between the reinforcing bar and the concrete as shown in Figure 2.15. He stated that crack patterns depend on the rib shape and properties of the boundary and surrounding concrete, and these cracks are not developed individually but mutual dependently. He also found that the concrete behaved similarly and ideally in the case that the rib angle was from 40 degrees and 105 degrees. On the other hand, in the case that rib angle was less than 30 degree, their bond behaviour was different. He also observed that concrete powders were piled up in front of the rib.

Moreover, Tepfers proposed an analytical model that can be useful for estimating the tensile stress distribution caused by the development of radial split cracks though he still paid attention to only smooth surfaced reinforcing bars. According to his results, when the test piece was pulled, due to the stress concentration, debonding at the boundary between the bolt and concrete started. After that, this debonding proceeded toward the edge direction along the reinforcing bar.

Figure 2.15 Cone shaped cracks and longitudinal splitting cracks

(Tepfers, 1973)

Farmer (1975) and Goodman (1989) took the initiative in the research of rock bolting and published valuable knowledge regarding rock bolting. Farmer conducted theoretical research on the fully grouted rock bolting, and proposed the exponential decay model along the rock bolt. Even though his theoretical modelling was limited to the case of the perfect bonding and elastic deformation, his modelling has been widely accepted by researchers (see Figure 2.16). In addition to this, he compared his analytical solution with a number of experimental results. In his experiments, rock bolts were installed into the concrete, limestone and chalk with resin. According to Figure 2.17, in the case of concrete, the relatively small axial force distribution obtained from analytical solution showed good agreement with the result of pull out test. On the other hand, the theoretical results with limestone and chalk didn't agree with those of experiments.

Figure 2.16 Stress and deformation in a grouted anchor
(modified after Farmer, 1975)

Figure 2.17 Comparison results of theoretical model with those of
experiment (modified after Farmer, 1975)

Goodman (1989) explained about the effectiveness of rock bolting regarding its ability for increasing the confining pressure. By adding the confinement, even though rocks are highly fractured or fissured, the majority of rocks become stronger if the rock blocks are interlocked with one other. As described in Subsection 2.1.3, if the rock can move freely without confinement, the rock sliding along their joint becomes possible. However, confining pressure makes it difficult for fractured or fissured rocks to move, whereby much more energy is necessary (see Figure 2.1). Therefore, the whole block system can produce more than ten times larger strength with the small increment of mean stress. This is one of the reasons why the rock bolting is useful in reinforcing the rock mass.

In the nineteen eightieth, these research had been extended by some researchers. Aydan (1989) conducted push-out tests, pull-out tests and shear tests to understand the reinforcement mechanism of rock bolting both qualitatively and quantitatively. In addition to these, he derived the constitutive law of rock bolting system analytically.

In his experiments, first of all, he did push-out tests and pull-out tests to obtain the bond strength or bearing capacity respectively and studied the effect of the bolt configurations and diameter of the borehole on their bearing capacities. The parameters studied were the ratio of the diameter of rock bolt to that of the borehole, surface shape of rock bolts and confining pressure. As a result, the load capacity obtained from push-out tests was 25 % higher than that from pull-out tests.

Aydan explained this reason by the so-called Poisson effect (In the case of smooth surfaced rock bolt, the radial stress becomes a compressive

in the push-out tests, while it tends to be a tensile in the pull-out tests). Furthermore, the load capacity of rock bolts with ribs was much higher than that of rock bolts without ribs. He mentioned that this was due to the geometric dilatancy caused by the differences of rock bolts surface, and probably it was one of the most important parameters to determine the load capacity of rock bolts. Based on observation results of failure mode, he classified its failure mode into following three categories.

- 1 . Failure along the bolt-grout interface
- 2 . Failure along the grout-rock interface
- 3 . Failure by splitting of grout and rock annulus

To analyse the failure mode in detail, he conducted several shear tests. Shear tests explained in Chapter 3 were carried out with the similar method to his tests. Parameters were the interval length between each of ribs (9 mm and 13 mm) and confining pressure (0.0, 1.0, 1.5 and 3.0 MPa). According to his results, he concluded the following three cracks were mainly observed as shown in Figure 2.18.

1. High angle tension crack (denoted as HATC in Figure 2.18)
2. Low angle combined tension-shear crack (denoted as LACTSC in Figure 2.18)
3. Shear crack (indicated as SC in Figure 2.18)

He explained that the high angle tension crack (hereafter called HATC) was generated at the tip of rock bolt rib, and penetrate toward inside the mortar. This crack was inclined at an angle between 60 and 80 degrees to the global shearing direction. On the other hand, low angle combined tension-shear crack (hereafter called LACTSC) initiated at the

edge of rock bolt rib as well as HATC, but the angle of LACTSC was between 10 and 35 degrees to the global shearing directions. The orientation of this crack starts to change toward the edge of the next rib as shown in Figure 2.18. According to the detail observation about the latter behaviour, this crack was generated due to the intense shearing. In addition to this, he summarised this kind of crack was the most important failure pattern.

Figure 2.18 Main cracks observed by Aydan during shear tests

(Aydan, 1989)

In the nineteen nineties, the research on cable bolts became more popular especially in Canada (Jean and Mark, 1996). Cable bolts are longer than rock bolts, and generally in the range from 3 to 15 m. The common types of cable bolts were summarised by Windsor (1992) as shown in Table 2.2. The force and displacement caused by the stressed cable bolt which is installed into the borehole with mortar materials are illustrated in Figure 2.19 (Hoek, 2006). As the cable bolt is tensioned, the radial displacement occurred from the boundary between the cable bolt surface and the mortar. This radial displacement enhances the confining pressure. As a result, the shear stress which can resist against the sliding of the cable is influenced by both this enhanced confining pressure and friction between the cable bolt and mortar materials.

Hyett, Yazici and Kaizer developed a theoretical model on the behaviour of cable bolt systems, which consists of the surrounding rock, grout materials and cable bolt itself. Hyett et al. (1992) conducted several pull-out tests using fully grouted cable bolt, and identified two failure modes based on their tests. The first one is the radial splitting of concrete surrounding the cable bolt. The other one is the shearing failure between cable bolt and the concrete. He summarised schematic images during pull-out tests shown in Figure 2.20. According to these figures, successive stages in the failure pattern during pull-out tests were divided into four stages. In the first stage, the load-displacement curve showed the mostly linear response. From the second stage, the failure patterns were depended on the radial confining pressure and stress decreasing might be influenced by the radial fracturing of the mortar and/or shear failure

Table 2.2 Typical types of cable bolts summarized by Windsor

(Windsor, 1992)

TYPE	LONGITUDINAL SECTION	CROSS SECTION
Multi-wire tendon (Clifford, 1974)		
Birdcaged multi-wire tendon (Jirovec, 1978)		
Single strand (Hunt & Askew, 1977)		
Coated single strand (Hunt & Askew, 1977)		
Barrel and wedge anchor on strand (Mathews et al, 1984)		
Swaged anchor on strand (Schmuck, 1979)		
High capacity shear dowel (Mathews et al, 1986)		
Birdcaged strand (Hutchins et al, 1990)		
Bulbed strand (Garford, 1990)		
Ferruled strand (Windsor, 1990)		

Figure 2.19 Forces and displacements associated with a stressed cable grouted into a borehole in rock (Hoek, 2006)

directing toward the global shearing direction. After that, as the axial displacement increased, the radial confining pressure was controlled by the biggest differences between the cable and mortar flutes. Finally, he concluded that the radial splitting mode is likely to occur under low confining pressure. On the other hand, the shearing failure mode is likely to happen as confining pressure increases.

Yazici and Kaizer (1992) proposed a bond strength model (BSM) to evaluate the load capacity of fully grouted cable bolts. In this model, the mechanical interlocking, which is the term for the shear resistance caused by bolt profile reacting surrounding materials, was considered with zig-zag surface model as shown in Figure 2.21. This surface created the dilation and radial movement when the debonding between the cable bolt

Figure 2.20 Schematic images of fixing material during pull-out tests

(modified after Hyett et al., 1992)

surface and mortar occurred.

In addition to the cable bolts, the study on the rock bolting was also continued. For example, Stillborg (1994) carried out a number of pull-out tests using many types of rock bolts. These rock bolts were installed two blocks of high strength reinforced concrete (uniaxial compressive strength = 60 MPa) which simulated a rock joint. Figure 2.22 shows a

Figure 2.21 Zig-zag surface bolt profile utilized for Bond Strength Model
(Kaiser et al., 1992)

Figure 2.22 Results of pull out test conducted by Stillborg
(Hoek, 2006)

typical result of pull-out tests. From these results, fully-grouted (both cement and resin) rebar rock bolts, which have been widely used all over the world, have high load capacity. However, they tend to fail easily when relatively large ground deformation occurs. On the other hand, swellex rock bolts and split set stabiliser, which have also been applied to a number of construction sites, can tolerate large deformation but can bear lower load than fully-grouted rebar rock bolts. Based on these experimental results, Li and Stillborg (1999) proposed interfacial shear stress (ISS) model to evaluate the rock bolt behaviour when the rock joint was opened. The deformation of the interface was considered with four stages which were elastic, linear softening, residual and debonding.

Over the last decade, further developments have been made on the rock bolting system. For example, the study utilising X-ray for observation of the failure pattern, variety of rock bolt shapes, etc. Obara et al. (2002) used X-ray CT scan for the detailed observation of failure patterns after the pull-out tests and concluded that the bond failure during pull-put tests was dependent on the bolt types and the bolt surface shape as shown in Figure 2.23. Furthermore, he summarised that radial cracks, which were generated from the surface of cable bolt, were caused by the internal pressure, and the effect of this internal pressure on the failure pattern was more dominant than that of twisting.

Kilic et al. (2003) conducted pull out tests using the rock bolts with cone-shaped lugs as shown in Figure 2.24 and revealed that the bolt shapes strongly affected the load bearing capacity and deformation behaviour.

Figure 2.23 Observation results after pull out tests using X-ray CT scan, modified from the original journal
(Obara et al., 2002)

Figure 2.24 Test pieces of pull out tests
(modified after Kilic et al., 2003)

According to his research, the bond bearing capacity obtained from the ribbed bar was 5.5 times higher than that of the smoothly surfaced bar. Besides, the bond capacity of the bolt with cone shape ribs was 27 % superior to that of the ribbed bar.

Aziz et al. (2008) analysed the influence of the rib spacing and summarised the optimum rib spacing was 37.5 mm. Moreover, he mentioned that as the rib spacing became closer, the failure mode of this rock bolt was similar to that of smoothly surfaced rock bolt which has a slightly large diameter. Cao (2012) proposed that the latest theoretical model which can estimate the rock bolt behaviour in detail. Researchers had also built some constitutive laws for the reinforcement mechanism. However, they could address only the major influence of rock bolts, grout and its interaction between boundary. On the other hand, the model produced by Cao can take the effect of the bolt shape on the rock bolt behaviour into account, even though his method seems to be able to apply to the only general shape of rock bolts such as a deformed reinforcing bar.

The aforementioned studies indicated that the supporting effect of a fully grouted rock bolt is significantly influenced by the load transfer capacity along the bolt-grout interface, and the load transfer capacity depends on many factors, including the shear strength and mechanical interlocking between the bolt and the bond material (Cao, 2012). In addition to these influential factors, the debonding failure that occurs along the interface between the rock bolt and the bond material is considered as the most critical behaviour for the reinforcement effect of the rock bolt. For example, Aydan (1989), Hyett et al. (1982) and Obara et

al.2) concluded that the interface debonding failure was one of the most important factors significantly affecting the reinforcement effect and support capacity of a rock bolt system. However, there is still a lack of comprehensive understanding on the detailed debonding process along the boundary between the rock bolt and mortar. Furthermore, the crack propagation into the bond material accompanied with the debonding process is still not clear.

2.4. Research review (numerical simulation)

Besides experimental studies, numerical investigations on the rock bolt system have been carried out by many researchers. Most of the numerical studies were developed based on the continuum approach, i.e. by incorporating the rock bolt element into the finite element method (FEM) or the finite difference method (FDM). According to the previous studies, the numerical modelling of rock bolts was strongly influenced by the studies on a reinforcing bar in a concrete (Shafer, 1975; Plauk and Hess, 1981; Noguchi 1981). In most of the numerical modelling of rock bolts, a rock bolt was modelled by a truss element or a beam element, which cannot accurately simulate the rock bolt behaviour installed in the borehole. This is because that the interaction between the rock bolt and the rock mass was reproduced only by the differences in the deformability of both materials (i.e., a rock bolt and rock mass). John and Dillen (1983) developed a new one-dimensional rock bolt element which can simulate the debonding failure between the rock bolt and the ground by utilizing the observation result of interface behavior observed from the laboratory

tests. Yap and Rodger (1984) proposed a rock bolt model that considered the effect of dilatancy. Aydan et al. (1985, 1989) proposed a model that considered even the kinking and bending behaviour of a rock bolt. The studies on rock bolt parameters (e.g., rib shape and rib space), which probably have a significant impact on the reinforcing effect of a rock bolt system, are rare in the literature. Li (2007) investigated the interaction behaviour between a steel bolt and a bond material using the FEM. However, the rock bolt interface behaviour is highly dependent on the rock mass and rock block deformation; it is probably difficult to represent the interface behaviour using continuum-based numerical methods as they cannot accurately model the fractured rock mass and effectively model the multiple crack propagation at the interface.

Recently, because of the significant improvement on the computer processing, discontinuous numerical methods such as the discontinuous element method (DEM), the discontinuous deformation analysis (DDA) and lattice model have been widely used in the field of rock mechanics. For example, Shang et al. (2018) built a particle-based DEM model taking account into the geometries of rock bridges and investigated the shear behaviour of incipient rock joints as shown in Figure 2.25. Nie et al. (2014) and He et al. (2017) proposed rock bolt models using DDA shown in Figure 2.26. The lattice model has been developed mainly in the field of concrete engineering to simulate crack propagation into the concrete material (Bolander and Saito, 1998). Recently, the lattice model has been applied to the estimation of rock fracturing (Ning et al., 2017; Asahina et al., 2017; Zhao and Xia, 2018). Other than these discontinuous numerical methods,

even continuum-based numerical methods (e.g., FEM) are theoretically applicable to simulate fracturing in intact rock. However, considerable efforts are required for the meshing and re-meshing processes. To overcome this problem, the conventional FEM has been improved using the partition of unity. The new method is referred to as the extended finite element method (XFEM) (Moes et al., 1999) and recently successfully applied to the study on rock fracturing (Xie et al., 2016, 2017). Moreover, these newly developed computational methods can be used even for 3D analysis.

Figure 2.25 Crack initiation, propagation and coalescence of an incipient joint with DEM (Shang et al., 2018)

Figure 2.26 Pull-out simulation with DDA

(He et al., 2017)

Figure 2.27 Lattice model of Brazilian tension test

(Ning et al., 2017)

Figure 2.28 Numerical simulation results of flaw inclination angle 45°

(Xie et al., 2016)

As described above, the rock bolt had been mainly modelled using FEM or FDM. However, these continuum based numerical simulation methods probably are not be able to reproduce the interface behaviour precisely, as it is difficult to simulate the crack initiation in the bond material and evaluate the influence of the crack propagation on the supporting effect of the rock bolt. Even though the dis-continuum based methods become frequently used for modelling the rock fracture, there is still a lack of numerical simulation model which can simulate the interface behaviour between the rock bolt and the bond material accurately.

2.5. Rock bolt under severe geological conditions

Over the last few decades, conventional tunnelling methods (i.e. the new Austrian tunnelling method, the drill and blasting tunnelling method) have been frequently used in the construction of tunnels/caverns under the severe geological conditions (e.g., high overburden, extremely shallow tunnel cover, unsymmetrical earth pressure/anisotropic stress condition and large cross-section). As a result, the role of tunnel support (i.e., rock bolt, steel set and sprayed concrete) has become more critical. Furthermore, in the case of tunneling under the extremely high overburden, it is necessary to select appropriate tunnel supports to prevent significant rock deformation caused by the rock burst or squeezing (Figure 2.29 and Figure 2.30).

Severe rock bursts can be observed when tunnels are excavated in hard rocks under the huge overburden (e.g., more than 1000 m depth), and rock bursts can be classified into three categories. The first category is referred to as 'strain burst'. When hard and massive rocks are excavated, the strain burst may occur because of the significant increase of the tangential stress around the excavated tunnel. The other two types of rock bursts are referred to as 'Fault-slip strain burst' and 'Fault-slip rock burst', which may be induced by the seismic waves as the fault may slip near the tunnel cutting faces. The fault-slip is typically generated because of the radial stress reduction after the tunnel excavation (Li, 2018).

Figure 2.29 Support system damaged by rock burst
(Kaiser and Cai, 2013)

Figure 2.30 Sheared-off steel arch by squeezing behaviour
(Rehbock and Jesel, 2018)

As defined by ISRM, the squeezing is defined as the large time-dependent deformation, mainly associated with creep which exceeds the shear strength of the rock mass (Barla, 1995), and the squeezing phenomenon can be observed especially during the tunnel excavation in the weak and soft ground (e.g., fault fracture zone) under high in-situ stress conditions. The weak rock around the excavated tunnel cannot resist the increased tangential stress, and the yielding zone is developed significantly, whereby the surrounding rock moves inward to the tunnel.

The amount of convergence and the extent of yielding zone are dependent on the ground conditions and in-situ stress conditions (Aydan et al., 1993; Barla, 2001). Both the rock burst and the squeezing phenomenon are the two main challenges in the tunnel construction which can result in significant delays in schedule and an increase in project cost (Steiner, 1996).

In the conventional rock bolt design, the strength capacity is the main consideration, and the rock bolts are expected to resist rigidly against the loading and the displacement during/after the excavation. However, in order to deal with the large deformation caused by the rock burst or the squeezing, it is necessary to consider not only their strength capacity but also their deformability. By allowing and controlling the deformation of the ground, the ground pressure acting on the support material such as rock bolts can be reduced, whereby the failure of support materials can be prevented (Hoek, 2006).

In the deep mining where rock burst may be induced, several energy-absorbing rock bolts (e.g., Cone bolt, Garford solid bolt, D Bolt),

which have both the high loading capacity as well as the large deformability, have been developed and applied successfully. Cone bolt is the earliest energy-absorbing rock bolt developed in South Africa (Jager, 1992; Ortlepp 1992). As shown in Figure 2.31(a), the cone bolt consists of a smooth steel bar with a flattened conical flaring forged on to the far end (Li, 2014), and it can withstand significant rock mass deformation by allowing the conically shaped anchor moving and crushing the grouting materials such as the mortar and the resin (Figure 2.31(b)). Garford solid bolt developed in Australia is comprised of a smooth steel bar, unique engineered anchor and steel sleeve at the far end of rock bolt (Varden et al., 2008; Li, 2014) as shown in Figure 2.32(a), where the inner diameter of the anchor is smaller than the diameter of the bar inside the sleeve. Once the rock deforms, the bar starts to extrude through the inside hole of the anchor, whereby it can absorb the enormous energy due to rock dilation (Figure 2.32(b)). D bolt developed in Norway consists of the smooth steel bar and multiple anchors as shown in Figure 2.33, which has been used in the field of deep mining in order to prevent the tunnel from collapsing because of the rock burst. D bolt can achieve both high loading capacity and deformability by the elongation of the steel bar between anchors (Li, 2010 and 2014).

(a) Photo of the cone bolt

(b) Supporting mechanism of the cone bolt

Figure 2.31 Cone bolt and supporting mechanism

(Li et al., 2014)

(a) Photo of the Garford solid bolt

(b) Supporting mechanism of the Garford solid bolt

Figure 2.32 Garford solid bolt and supporting mechanism

(Li et al., 2014)

Figure 2.33 D bolt and supporting mechanism

(Li, 2010)

Meanwhile, for the countermeasure against the squeezing ground, several yielding supports have been developed, such as the yielding steel set with a sliding mechanism as shown in Figure 2.34 (Barla, 2001; Hoek, 2006). For the sprayed concrete, the high deformable concrete which contains many glass beads and steel fibre as shown in Figure 2.35 (Arno et al., 2006) and the lining stress controller (LSC) which consists of multiple steel pipes in a concentric assembly (Schubert and Moritz, 1998) have been applied (Figure 2.36), in order to prevent brittle failure in the steel set and sprayed concrete. For the rock bolting system, abovementioned cone bolts and D bolts have also been employed to deal with squeezing deformation in the field of deep mining (Li, 2011). However, in civil engineering tunnels, the fully grouted rock bolts are still widely used even under the severe squeezing conditions as shown in Figure 2.37 (Fabbri, 2004), due to the requirement of the ultimate displacement of the tunnel wall as well as the final tunnel shape. In fact, as it is evident from

the load-displacement curves of cone bolts and D bolts (Li, 2014), these rock bolts can follow the rock movements to avoid the sudden tunnel collapse, but they are not designed to control the ultimate tunnel deformation. Hence, these rock bolts have not been widely used in civil engineering tunnels. Li (2017) concluded that the current rock bolting system for the squeezing behaviour in civil engineering tunnels has the incompatible problem, i.e. the ductile supports are applied only to the steel set and the sprayed concrete, and non-ductile supports are used for rock bolt. So there is a need to develop a type of energy-absorbing rock bolts with ductile characteristics.

(a) Assembly of a sliding joint

(b) Cross section detail

Figure 2.34 TH profile with sliding connection

(Barla, 2001; Hoek, 2006)

(a) Deformable lining with the hiDCon

(b) Detail of installed hiDCon-elements

Figure 2.35 High deformable concrete (hiDCon)

(Arno et al., 2006)

(a) A row of LSCs installed in a slot
in the shotcrete

(b) Cross section detail

Figure 2.36 Lining stress controller (LSC)

(Schubert and Moritz, 1998)

Figure 2.37 Yielding supports in Gotthard tunnel

(Davide, 2004)

As explained above, the energy-absorbing rock bolts are currently used in the deep mining as one of the countermeasure against the rock burst or squeezing. However, when it comes to using them in the civil engineering tunnel (e.g., transportation tunnel), it is necessary to pay more attention to the ultimate tunnel wall displacement and final tunnel shape. Therefore, it will be very meaningful to develop a new type of energy-absorbing rock bolt which can not only withstand large deformation but also control the final displacement.

2.6. Summary

From the result of literature review, the following three research gaps can be obtained. This study was mainly conducted in order to solve these research gaps.

-According to the previous studies based on the laboratory tests, the interface debonding failure was one of the crucial factors affecting the supporting effect of rock bolts. Several experimental studies have been conducted, however, there is still a lack of comprehensive understanding on the detailed debonding process along the boundary between the rock bolt and mortar. Furthermore, the crack propagation into the bond material accompanied with the debonding process is still not clear.

- According to the previous studies based on the numerical simulation, the rock bolt had been mainly modelled using FEM or FDM. However, it is not easy to reproduce the interface behaviour between the bolt and the

bond material with these continuum based numerical methods. In addition, it is more challenging to simulate the crack distribution and the accurate rock bolt behaviour after the crack initiation. Recently, the discontinuum based methods are frequently used for modelling the rock joints and fracture. However, there is still a lack of accurate numerical simulation model which can simulate the interface behaviour between the rock bolt and the bond material.

-In the deep mining, in order to cope with large deformation caused by the rock burst or squeezing, the energy-absorbing rock bolts are currently used. However, they are not employed in the civil engineering tunnel (e.g., transportation tunnel). This is probably because the ultimate tunnel wall displacement and final tunnel shape have to be paid attention to more than mining tunnels. Thus, a new type of energy-absorbing rock bolt, which can not only withstand large deformation but also control the final displacement, will be useful for the countermeasure against squeezing behaviour especially in the civil engineering field.

Chapter 3

Debonding process of the interface between rock bolts and bond material

Pull-out tests are commonly utilised for verifying the performance of rock bolts. When the pull-out test is carried out, the rock bolt is installed into the borehole drilled in the artificial rock or the steel pipe which simulates the borehole, and fixed with the mortar or the resin. From results of pull-out tests, the load bearing capacity and the deformability of rock bolts can be estimated. However, it is challenging to evaluate the interaction between the rock bolt and bond material such as the mortar and resin, and it is also difficult to observe the failure mode of the bond material. Both the interaction and failure mode strongly influence the load bearing capacity and the deformability of rock bolts. Therefore, Obara et al.(2002) utilised X-ray CT scan and Hyett et al.(1992) cut off the test pieces for analysing the particular mortar behaviour such as the crack initiation and propagation into the mortar. However, their observation methods enable us to observe test pieces only after pull-out tests have been finished, so there is little information available on the inspection results during pull-out tests. For this reason, Aydan (1989) employed several shear tests which simply simulated the rock bolt and the mortar, and observed detailed crack occurrence during shear tests. In his study, he dealt with the differences in shear behaviour caused by the confining

pressure and interval of rock bolt ribs. In this study, the effects of key rock bolt parameters (i.e., with/without rib, rib angle, mortar strength, confining pressure) on the interface behaviour and load capacity were investigated. The results of load – displacement curves and crack propagation obtained from laboratory tests can be utilised for the confirmation of the reliability of numerical simulations, which will be described in the next chapter. These shear test results can also contribute to better understand the reinforcing mechanism of rock bolting.

3.1. Outline of shear test

3.1.1. Shear test equipment

The shear test equipment employed in this study was originally developed to determine the shear strength parameters for rock joints. This experimental setup was prepared by ROCTEST and referred to as the portable shear box (PHI – 10). This portable shear box enables us to measure shear strength of rock samples or boring core samples whose diameters are less than 115 mm (or 4.5 inch). The main feature of this equipment is its normal load application system. This system is integrated with the pressure maintainer equipment which can keep the normal stress constant during shear tests, thereby preventing the rock sample from rotating even though the normal load is relatively small. Figure 3.1 shows the overall view of the portable shear box. The main composition of this equipment is briefly described as follows.

Figure 3.2 shows the longitudinal view of portable shear box assembly. The shape inside the shear box is cylindrical form. This shear box is

designed to apply the shear force to the exact shear plane of the test piece, and the position of the shear plane is controlled well and confirmed correctly. Figure 3.3 shows the pressure maintainer on normal load application system, to keep the pressure constant through shear tests. Before placing test pieces into the shear box, they are mounted with cement or other materials like gypsum. The mould where the test pieces are installed is comprised of two identical shaped forms and the locating clamp which can prevent test pieces from being out of position. Figure 3.4 shows the test sample mounted in the mould and Figure 3.5 shows the test sample held with the locating clamp. The frames are made of aluminium, and their side surfaces are composed of clear acrylic plastic so that the shear plane can be adjusted accurately.

Further details of specifications for the portable shear box, displacement transducer and pressure transducer are given in Table 3.1.

Figure 3.1 Overall view of the portable shear box (PHI – 10)

Figure 3.2 Longitudinal view of portable shear box assembly

Figure 3.3 Pressure maintainer on normal load application system

Figure 3.4 Test sample mounted in its mould

Figure 3.5 Test sample held with the locating clamp

Table 3.1 Specifications for the portable shear box, displacement transducer and pressure transducer

Portable shear box	Type	PHI-10 (ROCKTEST)		
	Hydraulic pump	Capacity	0-68,940	kPa
	Horizontal hydraulic ram	Capacity	89	kN
		Working force	44.5	kN
		Piston area	14.43	cm ²
		Working pressure	30,830	kPa
		Pressure gauge	0-35,000	kPa
	Vertical hydraulic ram	Capacity	133.5	kN
		Working force	44.5	kN
		Piston area	20.26	cm ²
		Working pressure	21,950	kPa
		Pressure gauge	0-35,000	kPa
	Pressure maintainer	Area ratio	27.04	
		Working pressure	811.8	kPa
		Pressure gauge	0-1,100	kPa

Displacement transducer	Type	CDP-10 (Tokyo Sokki Kenkyujo Co., Ltd.)		
	Vertical displacement	Capacity	10	mm
		Rated output	5mv±0.1%	
			(10000×10 ⁻⁶ Strain±0.1%)	
		Sensitivity	1000	×10 ⁻⁶ Strain/mm
	Type	CDP-25 (Tokyo Sokki Kenkyujo Co., Ltd.)		
	Horizontal displacement	Capacity	25	mm
		Rated output	6.25mv±0.1%	
			(12500×10 ⁻⁶ Strain±0.1%)	
		Sensitivity	500	×10 ⁻⁶ Strain /mm

Pressure transducer	Type	PGA-500KA (Kyowa Electronic Instruments Co., Ltd.)		
	Vertical pressure	Capacity	50	MPa
		Rated output	2mv±0.1%	
			(4000×10 ⁻⁶ Strain±0.1%)	
	Type	PGA-500KA (Kyowa Electronic Instruments Co., Ltd.)		
	Horizontal pressure	Capacity	50	MPa
		Rated output	2mv±0.1%	
			(4000×10 ⁻⁶ Strain±0.1%)	

3.1.2. Preparation for shear tests

First of all, the mould for fixing the test piece was built to create the test piece. Next, the test piece was set in the mould. At this time, the shear plane must be parallel to the direction of shear loading. In our test, the test piece was comprised of the steel part and mortar part which were simply to simulate the rock bolt and bond material. Therefore, the shear plane was the boundary between the steel and the mortar. After setting the test piece into the mould, gypsum which was to hold on the test piece was cast into this mould. After gypsum had been fixed, the mould was removed. Finally, the test piece with hardened gypsum was installed into the portable shear box. Figure 3.6 shows the preparation workflow.

3.1.3. Shear test procedure

Firstly, the confining pressure which corresponds to the vertical stress was kept constant. During the tests, the shear displacement, which was defined as the horizontal movement of the steel part, and shear force, which was defined as the horizontal loading on the steel part, were measured using a displacement metre and pressure gauge. The rate of shear loading was approximately 0.1 mm/s, and the confining pressure was varied between 2.0 MPa and 6.0 MPa. During the shear test, the crack initiation and propagation inside the mortar were observed in detail by incorporating a compact camera, which was installed in front of the test sample as shown in Figure 3.7. The schematic view of the shear test is shown in Figure 3.8.

Figure 3.6 Flow of preparation works

Figure 3.7 Shear test equipment

Figure 3.8 Schematic view of the shear test

3.2. Shear test cases

As described earlier, the fully grouted rock bolt was assumed as the simplified model shown in Figure 3.9. The test samples had three ribs in each steel block. The interval length between the ribs was 17.8 mm, and the rib height was 2 mm, which are the same as the specifications of typical rock bolts in Japan. The mortar height was 12mm.

Figure 3.9 Image of simplified rock bolt model (Upper right: Rib angle=30°, Lower left=60°, Lower right=90°)

When the rock bolt whose diameter is 26 mm is installed into the borehole (Diameter = 50mm), the mortar length can be expressed as the following;

$$(50mm - 26mm)/2 = 12mm \quad (3.1)$$

Table 3.2 shows laboratory test cases, and in total twelve tests were carried out with three main parameters: rib angle 0° (i.e., without ribs), 30°, 60°, 90° , a mortar strength of 10 MPa and 30 MPa and a

confining pressure of 2 MPa, 4 MPa and 6 MPa. The mortar strength of 10 MPa is the average strength of the mortar after 24 hours curing. For the confining pressure, the research result by Aydan et al. (1995) was taken into account. They suggested estimating the effective confining pressure acting on the rock bolt in tunnel excavation sites as shown in Figure 3.10.

Table 3.2 Laboratory test cases

Case No.	Rib angle (deg)	Mortar strength (MPa)	Confining pressure (MPa)
01	0	10	2.0
02	0	10	4.0
03	0	10	6.0
04	30	10	2.0
05	60	10	2.0
06	90	10	2.0
07	30	10	4.0
08	60	10	4.0
09	90	10	4.0
10	30	30	2.0
11	60	30	2.0
12	90	30	2.0

Figure 3.10 Estimation model of interface shear strength in boreholes
(Aydan et al., 1995)

In fact, since the effective confining pressure acting on the rock bolt is related to the rock deformation modulus, Aydan et al.(1995) revealed that the effective confining pressure could be derived from the mathematical expression with the shear test results and the radial response of the surrounding medium around the borehole. For example,

in the case of 45° of the rib angle, the effective confining pressure can be written as:

$$\sigma_n = \frac{h}{\eta} \left[-1 + \sqrt{\frac{E}{1+\nu} \cdot \frac{\eta h}{aL}} \right] \quad (3.2)$$

where h is the height of each rib = 0.002 m, ν is Poisson ratio = 0.25, η is the sealing value for changing the unit = 0.001, L is the interval length of each rib = 0.0178 m and a is the diameter of the borehole = 0.05 m.

From Eq. (3.2), the relationship between the effective confining pressure and rock deformation modulus can be obtained shown in Figure 3.11. Since the upper limit of typical rock deformation modulus can be considered as 10 GPa, the confining pressure for the shear tests was set between 2 MPa and 6 MPa.

Figure 3.11 The relationship between the effective confining pressure and rock deformation modulus

3.3. Shear test results

3.3.1. Comparison of crack distribution

Figure 3.12 shows the photos taken at the end of the shear tests under a confining pressure of 2.0 MPa. As can be observed from the photo, there were no major cracks in the mortar for the case of rock bolt without ribs during the shear tests. On the other hand, some cracks were observed in the mortar for rock bolts with ribs. Depending on the mortar strength, different crack patterns were observed for the rock bolts with ribs. In order to observe the different crack patterns carefully, the digital particle image velocimetry (DPIV) method and the high resolution 3D surface scanner were utilised for the observations. A brief introduction of both methods is presented here.

Figure 3.12 Photos after shear tests without ribs under a confining pressure of 2.0 MPa

Digital Particle Image Velocimetry (DPIV)

Digital particle image velocimetry (DPIV) is non-intrusive velocity measurement method developed to visualize the flow mapping quantitatively and qualitatively. This technic has been also applied in the field of geotechnical engineering (White et al., 2003). In this research, GUI-based open-source tool (Thielicke and Stamhuis, 2014) was employed to analyze the mortar behaviour in detail during the shear tests. Fundamental analysis procedure is as follows: i) divide the photo ($t = t_1$) into small patches; ii) cross-correlate these small patches ($t = t_1$) with the photo taken after certain time ($t = t_2$); iii) The highest peak in the correlation result indicates the displacement vector of these small area (Figure 3.13). The cross-correlation is a kind of statistical pattern matching method. By applying this method, the most probable displacement vector between the two photos can be estimated.

Figure 3.13 Image manipulation during PIV analysis (White et al., 2003)

High resolution 3D surface scanner

In order to observe the test samples in particular the mortar surface after shear tests, the software David laser scanner was utilised. This software can produce a detailed three-dimensional model of scanned object by using a general web camera and a laser pointer as shown in Figure 3.14 (Winkelbach et al., 2006). Recently, by replacing laser pointer to a projector light, better results can be obtained more quickly. In the latest system, the various light patterns, which consist of horizontal and vertical with different width strips, are projected to a scanned object. By capturing the distorted light patterns with the camera, three-dimensional coordinates can be calculated. The accuracy of this scanning method is up to 2 % of a scanned object (Novak et al., 2014), and moreover previous studies concluded that its resolution was between 0.06 mm and 0.8 mm (Friess, 2012).

Figure 3.14 Measurement conceptual image with a camera and laser pointer (Winkelbach et al., 2006)

Figure 3.15 and Figure 3.16 show the shear test results, in which the DPIV results, the 3D scanning results, the photos taken when the horizontal displacement was 2.0 mm. Figure 3.15 are results for the low-strength mortar (10 MPa), and Figure 3.16 are the results for the high-strength mortar (30 MPa).

First, DPIV analyses are conducted using the photos taken just after the crack initiation. In the case of low-strength mortar, only small amount of mortar between ribs moves to the shearing direction. On the other hand, the result with high-strength mortar shows that the upper mortar also moves to the right direction. From these results, two areas, one area with large displacement vector, and the other area with small displacement vector can be identified. The difference of the displacement vectors indicates that cracks are initiated at the boundary which divided the mortar into two areas. Second, the 3D surfaces were scanned using a camera and projector after the mortars were removed from the test samples. From the scanned result with low-strength mortar, there are no visible cracks in the mortar, and the scanned surface is found to be almost flat except the boundary between the steel and mortar (shown as blue line). On the other hand, the scanned result with high-strength mortar shows the uneven surface which is comprised of several undulated lines (shown as dash line). By observing the location and angle of these undulated lines, it is found that they start from the edge of each rib and their angle is ranged from 20° and 30°. Moreover, these undulated lines are almost matching the boundary observed by DPIV analysis.

(a) DPIV result

(b) 3D scanning result

(c) Photo (Displacement = 2.0 mm)

Figure 3.15 Detailed observation results with Low-strength mortar (UCS = 10 MPa)

(d) DPIV result

(e) 3D scanning result

Figure 3.16 Detailed observation results with High-strength mortar
(UCS = 30 MPa)

In summary, in the case of low-strength mortar, low angle cracks ($0 - 10^\circ$) are generated from the edge of the ribs, and the mortar is sheared horizontally. In contrast, in the case of high-strength mortar, the mortar is failed brittly, and horizontal shear cracks as well as inclined cracks ($20-30^\circ$) are observed. In terms of the crack distribution, the influence of the rib angle (30° , 60° , 90°) and confining pressure was not significant.

3.3.2. Comparison of load – displacement curve

Figure 3.17 shows the typical relationship between the shear stress and horizontal displacement, which is referred to as the load–displacement curve. For rock bolts without ribs, both the peak strength and the residual strength became higher as the confining pressure was increased. Figure 3.18 shows the initial cohesion “ c_i ” and internal friction angle “ ϕ_i ” between the steel and mortar material. The residual cohesion “ c_r ” and residual friction angle “ ϕ_r ” were also obtained. From this figure, “ c_i ”, “ ϕ_i ”, “ c_r ” and “ ϕ_r ” can be found as 0.5 MPa, 39.5° , 0.0 MPa and 31.3° respectively. These values form the basic material parameters for DDA simulations. On the other hand, the shear stiffness, which is defined as the ratio of the peak shear strength to the horizontal displacement, was not significantly affected by the confining pressure.

Three different sets of test conditions were adopted for rock bolts with ribs: a) different rib angles (30° , 60° , and 90°) under 2 MPa confining pressure; b) different rib angles (30° , 60° , and 90°) under 4 MPa confining pressure; and c) three cases of rock bolts with ribs and a high strength mortar (30 MPa). The key observations based on Figure 3.17 are as follows.

(a) Without ribs

(b) With ribs (low confinement)

(c) With ribs (high confinement)

(d) With ribs (high mortar strength)

Figure 3.17 Load – displacement curve (Laboratory tests)

(RA=Rib angle, MS=Mortar strength, CP=Confining pressure)

Figure 3.18 Relationship between shear stress and normal stress

With / without ribs

As shown in Figure 3.17(a), the shear stress of the rock bolt without ribs increased linearly until the peak strength. In other words, the shear stiffness was found to be constant. Meanwhile, the curves for the rock bolts with ribs showed that the shear stiffness slightly decreased as the shear stress came close to the peak strength, as shown in Figure 3.17(b). In the case of the rock bolts without ribs, the sliding behaviour merely occurred along the boundary between the rock bolt and the mortar. On the other hand, in the case of the rock bolt with ribs, the result was influenced by the deformability of the bond material because each rock bolt rib was interlocked with the mortar during the shear tests.

In addition, different post failure modes after the peak strength can be confirmed. As shown in Figure 3.17(a) and (b), the shear stress without ribs reached the residual value earlier than those with ribs. This can be explained by the observations, which showed that each rib

continued to provide an interlock with the mortar, even after the peak strength in the case of the rock bolts with ribs. In the case of the rock bolt with ribs, it was observed that the number of cracks increased after the peak, and the residual shear strength was finally obtained when horizontal shear planes were formed between the ribs.

Influence of the rib angle

Although the impact of the rib angle on the peak strength was insignificant, the shear stiffness with a 30° rib angle was slightly smaller than the other angles regardless of the mortar strength (see Figure 3.17 (b) and (d)). During the laboratory tests, it was confirmed that the slip behaviour along the front surface of each rib was triggered more easily as the rib angle became lower. Due to this slip behaviour, a large horizontal displacement was initiated, and the shear stiffness slightly decreased. This interpretation can be proven by the results with a high confining pressure shown in Figure 3.17(c). As the confining pressure increased, the resulting variations with different rib angles were insignificant. In other words, the effect of interlocking at the bolt rib–mortar boundary was increased by the confining pressure, whereby the significant slip behaviour was not observed, even in the case with a lower rib angle.

Influence of the confining pressure

In the case of the rock bolt without ribs, even though the confining pressure was increased to 4 MPa, the shear stress increased linearly until the peak strength, similar to the results obtained under a confining

pressure of 2 MPa. Moreover, the shear stress also tended to decrease to the residual value, similar to the results under 2 MPa of confining pressure as shown in Figure 3.17(a). On the other hand, in the case of the rock bolt with ribs, the shear stiffness decreased considerably when the shear stress approached the peak strength. Additionally, the horizontal displacement, when the shear stress reached its residual value, also increased significantly, as shown in Figure 3.17(c). These observations could be due to the effect of interlocking at the bolt rib–mortar boundary, where a confining pressure strengthened the resistance of the mortar against the movement of each rock bolt rib considerably before and after the peak strength. As a result, the number of cracks in the mortar also increased.

As can be seen clearly in the result of Case07, the shear stress suddenly dropped just before reaching the residual strength. In fact, during the laboratory tests, some rock bolts slid slightly within a very short duration just after their peak strength. This sliding behaviour might be dependent on how the cracks are distributed in the mortar after the peak strength. Moreover, it was also observed that the shear stress started to increase again, because the new cracks were initiated, and the crack distribution was changed gradually, whereby the rock bolt rib interlocked again with the mortar at the new bolt rib and mortar boundary.

Influence of the mortar strength

As described in the previous sections, in the case of the rock bolt with ribs, the shear stiffness slightly decreased as the shear stress

approached the peak (see Figure 3.17(b)). However, the results with the high-strength mortar (UCS=30 MPa) showed that this trend became insignificant, as shown in Figure 3.17(d). These differences can undoubtedly prove that the decrease of shear stiffness was due to the effect of the mortar deformability. In short, as the mortar strength increased, its deformability decreased. Therefore, the mortar did not deform significantly before it reached its peak strength, which was the main reason why the shear stiffness did not change considerably in the case of a high-strength mortar.

For the horizontal displacement generated after the peak, a slightly larger displacement could be observed with the high-strength mortar than the cases with the low-strength mortar. However, the impact of the confining pressure was more significant. Meanwhile, in the case of the high-strength mortar, the shear stress continued to gradually reduce, even after the shear stress approached the residual strength, which may be due to the differences in the crack distributions, as shown in Subsection 3.3.1. With the high-strength mortar, many inclined cracks were generated. Therefore, after the peak, the mortar might have been damaged considerably compared to the rock bolt with the low strength mortar, creating a possibility for the damaged mortar to affect the residual strength.

3.3.3. Further discussions

Figure 3.19 shows the extracted data from Figure 3.17 to better compare the influence of the main rock bolt parameters (rib angle,

confining pressure and mortar strength). As shown in Figure 3.19(a), the effect of the rib angle was insignificant, as the crack initiation and mortar failure pattern were not substantially affected by the different rib angles. On the other hand, the results of the shear tests were highly dependent on the confining pressure and the mortar strength. Figure 3.19(b) represents the impact of the confining pressure, which confirmed that both the peak strength and residual strength increased as the confining pressure became large. This effect was the same regardless of existence of ribs. Figure 3.19(c) shows the influence of the mortar strength. When a low-strength mortar was utilised, low angled cracks dominated. Meanwhile, when a high-strength mortar was used, high angled cracks ranging 20° and 30° were observed, which was consistent with previous findings by Aydan (1989), and the mortar failed brittly. These differences in the mortar failure patterns were obviously caused by the mortar strength. Therefore, it can be concluded that the reinforcing effect of the rock bolt increases as the mortar strength increases. However, it is noted that the residual strength decreased, possible reason is that the mortar was damaged with the inclined cracks and failed brittly before horizontal shear cracks could dominate, whereby the ultimate residual strength was slightly decreased.

Figure 3.19 Influence of main rock bolt parameters

(RA=Rib angle, MS=Mortar strength, CP=Confining pressure)

3.3.4. Summary of shear tests

Table 3.3 show the summary of these shear tests. The following can be concluded from all shear test results.

Rock bolt without ribs

- As the confining pressure increased, both the peak strength and residual strength became high.
- On the other hand, the shear stiffness was not affected so much compared to the peak and residual strength.
- From obtained test results, the boundary condition between the rock bolt and mortar was calculated as follows;
  - $c_i = 0.5 \text{ MPa}$ and $\varphi_i = 39.5^\circ$
  - $c_r = 0.0 \text{ MPa}$ and $\varphi_r = 31.3^\circ$

Rock bolt with ribs

- The peak strength and residual strength with steel ribs were approximately 50% larger than those results without the rib.
- At the same time, the shear stiffness was decreased.
- Differences of the rib angle (30°, 60°, 90°) hardly influenced both the load – displacement curve and crack penetration.
- When the mortar strength was relatively high (30 MPa), inclined cracks were generated, and the mortar was failed brittly.
- The peak strength and shear stiffness of high-strength mortar (30 MPa) was increased, while the residual strength was slightly decreased.

- As the confining pressure was increased, the peak strength and residual strength were also increased.
- This effect of the confining pressure was similar to that of results without ribs.

Table 3.3 Image of dominant mortar failure patterns under 2.0MPa
confining pressure

Parameters		Mortar Failure Pattern	
Rib Angle	0°		
	30°		
	60°		
	90°		
Mortar Strength	10MPa		
	30MPa		

— :Crack

3.4. Summary

3.4.1. Simplified rock bolt model without ribs

First, the results without ribs were found to be highly repeatable. Second, these test results were highly dependent on the boundary condition between the bolt and the bond material since no cracks were observed inside the mortar, and also dependent on the increase of confining pressure. By taking above results into account, differences in mortar strength probably will not affect strongly.

3.4.2. Simplified rock bolt model with ribs

According to the results, differences of the rib angle (30°, 60°, 90°) did not make a significant impact on the load – displacement curves. It is concluded from experiment results that this is because the crack occurrence and mortar failure patterns were not affected by the rib angle substantially. On the other hand, it was observed that the mortar behaviour is highly dependent on the mortar strength. When low-strength mortar was utilised, low angled crack inclined at an angle of about 0° - 10° was observed. This low angled crack is consistent with previous findings by Aydan (1989). Meanwhile, when high-strength mortar was used, high angled crack which is inclined at an angle ranging 20° and 30° was observed, and also the mortar was failed brittlely. These differences in mortar failure patterns were obviously caused by the mortar strength. It is no wonder that the reinforcing effect of rock bolt increases as the mortar strength becomes high. Actually, compared to results with low-strength mortar, the peak strength and shear stiffness were improved.

However, it is noteworthy that the residual strength of higher strength mortar was slightly decreased, possibly because the mortar was damaged with inclined cracks and failed brittlely before horizontal shear cracks were dominant, whereby the ultimate residual strength was decreased. These impacts of different crack patterns could be obtained by detailed observation results during debonding process using image analysis and 3D scanning data. These findings influence strongly on energy absorbing rock bolts since these rock bolts are expected to tolerate both large load and ground movement even after peak strength. In addition, the relationship between the rib shape and mortar strength becomes more important in terms of mortar failure pattern, therefore it is recommended that various parametric studies should be examined with the numerical method which can simulate the crack propagation in detail.

Chapter 4

Numerical modelling using discontinuous deformation analysis

The purpose of this chapter is to analyse in detail on the interaction between the rock bolt and the mortar, and the mortar behaviour during shear tests using the discontinuous deformation analysis (DDA) which is one of dis-continuum based analysis methods. First, the shear tests were simulated to verify the accuracy of the DDA numerical simulation model by comparing the simulation results with laboratory tests. As a next step, with the accurate DDA numerical simulation model, the effects of the main rock bolt parameters (i.e., rib angle, rib height, rib space and rib shape) on mortar-bolt interface behaviour were evaluated. These parameters are difficult to consider in laboratory tests. To the best of the author's knowledge, no research has yet been carried out to find out the support mechanism of rock bolting using the numerical methods which can simulate the crack propagation and the behaviour after many cracks have occurred in the bond material. This kind of study will contribute to the better understanding on the supporting mechanism of rock bolts. In the early part of this chapter, the background, features and application examples of DDA were illustrated.

4.1. Introduction to discontinuous deformation analysis

The DDA was originally developed by Shi (1988) to simulate the mechanical behaviours of rock mass structure. Rock mass has many faults, cracks and joints, thereby being representative discontinuous materials. However, the DDA itself is a general-purpose numerical method, and it can be utilised not only for rock mass structure but also for multiple purposes including the contact or collision problems and behaviour of granular materials. Therefore, the DDA method can be the suitable tool for simulating shear test with the simplified rock bolt model, which was described in Chapter 3 because many cracks occurred and penetrated into the mortar material during shear tests.

4.1.1. History of discontinuous deformation analysis

The finite element method (FEM) has been a widely used numerical analysis for simulating the behaviour of continuous materials. To conduct the stability analysis of rock mass including discontinuities by using the FEM, Goodman et al.(1968) firstly developed so-called “joint element”. This joint element enabled the numerical model to represent rock separation and sliding, and therefore was considered as the most advanced numerical method. However, it was only available when deformation was infinitesimal deformation. Consequently, it was found to be insufficient to simulate rock mass behaviour after its failure since large and rigid body deformation had to be taken into account. Hence various kinds of methods were developed and proposed, but it has not been

successful to evaluate the rock mass behaviour after the failure using the FEM.

In such a situation, from the different point of view, the distinct element method (DEM) was proposed by Cundall(1971). In this latest numerical method, the block contact and collision were modelled with the spring and dashpot, and Newton's second law was utilised. Calculation method was simple and computation time was also quick, therefore the DEM has been utilised for a variety of fields, and there is a commercially produced code used among engineers. Furthermore, general-purpose code using circular or sphere blocks have been developed successfully. However, the uniqueness of solution cannot be ensured because of finite difference methods and also the unclear definition on block contact points. For example, as shown in Figure 4.1, it is hard to explain the theoretical reasons for the relationship between the spring, dash pot and real physical phenomenon, so the trial-and-error analysis becomes necessary. Moreover, it becomes harder to obtain the stable solution as the motion of blocks become faster.

The DDA was originally intended to identify the block movement from multiple measurement results in the rock mass including discontinuities, i.e. the so-called back analysis. The number of measurement points is very few due to budget constraint generally. Therefore the results of measurement displacements are not likely to agree with those of numerical simulation especially when there are many joints in rock mass. In such a case, DDA, which can build rock mass model based on geological information and interpret the measurement results

by taking into account the rock block movement, becomes the effective tool. DDA can treat blocks as elastic materials and evaluate energy as a quadratic expression by introducing penalty functions shown in Figure 4.2. By doing so, the degree of freedom is never changed, and energy conservation law can be ensured. Hence, both the uniqueness of solution and convergence can also be assured.

Figure 4.1 DEM(Cundall, 1971)

Figure 4.2 DDA (Shi, 1988)

Figure 4.3 shows the classification for representative discontinuous analyses which have been developed since about 50 years ago. From the viewpoint of elastic theory, discontinuous analyses were classified under the assumption of rigid body, infinitesimal and large deformation.

In the assumption of rigid body deformation, the structure never deforms. By using this assumption, block theory was developed by Goodman and Shi (1985). This method enables us to simulate the possibility of block movement which is located on the rock slope or excavation surface by processing mathematically the combination of three-dimensionally distributed faults and joints. For instance, results based on the block theory can be utilised for designing the falling rock preventive countermeasure.

In the assumption of infinitesimal deformation, Goodman et al. (1968) developed the joint element. The intact rock is modelled with finite elements, and the discontinuous surface is made with joint elements in his method. The generic name for these types of special elements is thin layer element, and a variety types of thin layer elements have been proposed after Goodman developed the joint element.

In the assumption of large deformation, Shi (1991) proposed Manifold method in addition to the above mentioned DEM and DDA. Manifold method shares features of both the FEM and the DDA.

Figure 4.3 Classification for representative discontinuous analyses

4.1.2. Main characteristics of discontinuous deformation analysis

In the field of rock mechanics, the DDA has been recognised as a useful method which enables to simulate large deformation and failure behaviour of rock blocks both statically and dynamically. The DDA has mainly following features.

- Similar to the FEM, the DDA utilises the “Principle of stationary potential energy”, and therefore the uniqueness of solution can be ensured.
- Strains, rigid body displacement and rigid body rotation, which can be defined at the centre of gravity in the arbitrary individual rock block, are treated as unknown variables. Then, by assuming strain is constant in each block, simultaneous linear equations are formed and solved.
- By inputting respective block geometry, loading condition, material properties, stiffness and strength parameters at block interfaces, and

also by preventing one block from penetrating other blocks using penalty method at block interfaces, the stress, strain, displacement and sliding of all blocks can be calculated.

- The excavation is simulated step-by-step with small time increment, and it becomes easier to introduce the nonlinear constitutive law to each block.

Main features of DDA are to conduct the large deformation analysis of both elastic and nonlinear material. The derivation of the equation based on a complete kinematic theory, their numerical processing, the precise mechanism of energy loss and establishment of exact equilibrium equations between each block, are also considered as the essential characteristics. A brief introduction of the DDA theory is presented in the subsection.

In the DDA model, the displacements at any point (x, y) inside a block are represented as follows:

$$\begin{pmatrix} u \\ v \end{pmatrix} = \begin{pmatrix} 1 & 0 & -(y - y_0) & (x - x_0) & 0 & (y - y_0)/2 \\ 0 & 1 & (x - x_0) & 0 & (y - y_0) & (x - x_0)/2 \end{pmatrix} \begin{pmatrix} u_0 \\ v_0 \\ r_0 \\ \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{pmatrix} \quad (4.1)$$

where (u_0, v_0) is the rigid body translation at a specific point (x_0, y_0) within the block, r_0 is the rotation angle of the block with respect to (x_0, y_0) , ε_x and ε_y are the normal strains in x and y directions, respectively, and γ_{xy} is the shear strain.

Writing Eq. (4.1) in a generalised form, Eq. (4.2) can be obtained.

$$\begin{Bmatrix} u_i \\ v_i \end{Bmatrix} = \begin{bmatrix} t_{11} & t_{12} & t_{13} & t_{14} & t_{15} & t_{16} \\ t_{21} & t_{22} & t_{23} & t_{24} & t_{25} & t_{26} \end{bmatrix} \begin{Bmatrix} d_{1i} \\ d_{2i} \\ d_{3i} \\ d_{4i} \\ d_{5i} \\ d_{6i} \end{Bmatrix} \quad (4.2a)$$

$$\text{or} \quad [u_i] = [T_i]\{D_i\} \quad (4.2b)$$

For a system with n blocks, the simultaneous equilibrium equations have the following form (Shi, 1988):

$$\begin{bmatrix} K_{11} & K_{12} & \dots & K_{1n} \\ K_{21} & K_{22} & \dots & K_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ K_{n1} & K_{n2} & \dots & K_{nn} \end{bmatrix} \begin{bmatrix} D_1 \\ D_2 \\ \vdots \\ D_n \end{bmatrix} = \begin{bmatrix} F_1 \\ F_2 \\ \vdots \\ F_n \end{bmatrix} \quad (4.3a)$$

$$\text{or} \quad [K_{ij}]\{D_i\} = [F_i] \quad (4.3b)$$

where $K_{ij}(i \neq j)$ is a 6×6 sub-matrix, representing the contacts between blocks i and j, $K_{ij}(i = j)$ is the block material stiffness matrix, D_i and F_i are 6×1 sub-matrices where D_i represents the deformation variables $(u_0, v_0, r_0, \varepsilon_x, \varepsilon_y, \gamma_{xy})$ and F_i represents the loading on block i distributed to the six deformation variables.

4.1.3. Application of discontinuous deformation analysis

DDA has broad utility. Therefore it can be applied to a wide range of fields. For example, not only rock mechanics but also the contact

problem of machine components like a gear, collision of materials, dynamic motion, granular material flow can be simulated using DDA. In this subsection, several application results are illustrated. Figure 4.4 shows that failure of a Brazilian disc with an initial hole by DDA and experiment (Zhao et al., 2013). This result demonstrated that crack length, width and angle observed with the numerical simulation were consistent with those of the laboratory test. Figure 4.5 shows the simulation result of rock cavern excavating in the rock mass with two joint sets and the fault. According to this result, the rock block near the left wall of the cavern was falling out toward to the tunnel after the excavation. Thus, it is known that DDA enables us to evaluate the behaviour of discontinuity like rock joints precisely.

Figure 4.4 Failure of a Brazilian disc with an initial hole
by DDA (Left) and experiment (Right)
(Zhao et al., 2013)

Stress contour

Time history curve

Figure 4.5 Simulation result of rock cavern excavating in rock mass
with two joint sets and the fault
(Zhao et al., 2013)

4.2. Numerical simulation of shear tests

4.2.1. Simulation model

First of all, the results of laboratory shear tests which were described in Chapter 3 were simulated. In this section, the simulation results of crack propagation and load-displacement curve have been evaluated in comparison with those test results. Figure 4.6 shows the simulation mesh for the DDA model. The simulation mesh is comprised the mortar model and the rock bolt model. The vertical displacement of rock bolt is fixed at the bottom, and the shear test can be reproduced by loading the force horizontally at the right edge of the rock bolt. Meanwhile, the confining pressure is loaded at the top of rock model. Properties of the rock bolt and the mortar are summarised in Table 4.1 and Table 4.2, respectively. Two types of mortar, low-strength mortar (UCS = 10 MPa) and high-strength mortar (UCS = 30 MPa) were assumed and their properties were decided using tri-axial test results (see Figure 4.7), splitting tensile test results and past literatures (Lim et al., 1998; Kiyohara et al., 1999; STC, 2002; JSCE, 2010). The rock bolt properties were referred to design values established by Japan Railway Construction, Transport and Technology Agency. In the 2D DDA model, the mortar was modelled by sub-blocks (Ning et al., 2011). The number of blocks in the mesh was between 2000 and 2500, and the average block size in the mortar was approximately 1.0 mm². The boundaries between sub-blocks were called ‘artificial joints’ assigning with friction angle ϕ , cohesion c and tensile strength σ_t . By the Mohr-Coulomb failure criteria with a tension cut off (see Figure 4.8), fracturing might occur along the pre-set artificial

joints between blocks. Then, the corresponding joint strength was changed to the residual friction angle ϕ' and residual cohesion c' , while tensile strength was set to zero.

Simulated model without rib

Simulated model with ribs

Figure 4.6 Simulation mesh by DDA

Figure 4.7 Tri-axial test results (left: UCS=30MPa, right: UCS=10MPa)

Table 4.1 Input data for rock bolt

Material	Parameter	Value
Rock bolt	Unit weight (kN/m ³)	78
	Young's modulus (GPa)	200
	Poisson's ratio	0.3
Rock bolt/mortar boundary	Friction angle (°)	35 / 30
	(Initial / residual)	
	Cohesion (MPa)	0.5 / 0.0
	(Initial / residual)	

Table 4.2 Input data for mortar model

Material	Parameter	Value
Mortar (Low strength: UCS=10MPa)	Unit weight (kN/m ³)	23
	Young's modulus (GPa)	3.4
	Poisson's ratio	0.2
	Friction angle (deg)	37.5 / 40
	(Initial / residual)	
	Cohesion (MPa)	4.5 / 1.25
	(Initial / residual)	
	Tensile strength (MPa)	1.7 / 0.0
	(Initial / residual)	
Mortar (High strength: UCS=30MPa)	Unit weight (kN/m ³)	23
	Young's modulus (GPa)	7.3
	Poisson's ratio	0.2
	Friction angle (deg)	38.5 / 42.5
	(Initial / residual)	
	Cohesion (MPa)	9.5 / 3.0
	(Initial / residual)	
	Tensile strength (MPa)	1.8 / 0.0
	(Initial / residual)	

Figure 4.8 Mohr-Coulomb criterion with a tension cut off in DDA

4.2.2. Simulation cases

Table 4.3 shows DDA simulation cases which were carried out in order to reproduce the results of shear tests. In this section, the effect of the confining pressure on the simulation results for bolt without rib and the effect of the mortar strength on the simulation results for bolt with ribs were mainly discussed. The mortar strength was found to be one of the most crucial parameters which affected crack distribution in the mortar in the case of bolt with ribs. After the validation of the DDA model by comparing with the experiment in this section, the effect of the rock bolt configurations (i.e., rib angle, rib height, rib space and rib shape) will be studied in the next section.

Table 4.3 DDA simulation cases

Case No.	Rib angle(°)	Mortar strength(MPa)	Confining Pressure(MPa)
Case 01	0	30	2.0
Case 02	0	30	4.0
Case 03	0	30	6.0
Case 04	60	10	2.0
Case 05	60	30	2.0

4.2.3. Numerical Simulation results

Rock bolt without ribs

Before conducting the numerical simulation, sensitivity analyses for critical DDA parameters such as step time, displacement ratio and spring value were carried out. The spring value represents the normal stiffness value in contact. The dimension of spring stiffness in DDA is N/m. In all simulations, the ratio of the normal stiffness to horizontal stiffness was set to 2.5.

According to results of sensitivity analyses, when step time was ranged from 0.00001 to 0.001s, the influence was insignificant. Similarly, when displacement ratio was varied from 0.00005 to 0.005, the effect was also trivial. On the other hand, as can be seen from Figure 4.9, it could be confirmed that the load – displacement curve, especially shear stiffness before the peak strength, was strongly dependent on spring value. Therefore spring value is found to be an essential parameter that affects the accuracy of the simulation.

In the following simulation results, the suitable spring value, which was selected based on the sensitivity analyses, was employed. Figure 4.10 shows the DDA results without ribs after the shearing simulation (SS=30). To allow an accurate comparison of results, the picture of the test piece which was taken after the shear test was also illustrated. According to this photo, no cracks were observed in the mortar during the laboratory test. As a result, it could be found in the previous chapter that all of the test pieces were sheared along the boundary between the rock bolt and the mortar. For the DDA results, there were

also no cracks in the mortar model at all. Consequently, the DDA results without ribs are found to be consistent with results of laboratory tests.

Figure 4.11 shows the load – displacement curve obtained from numerical simulations and the peak strength, residual strength, horizontal displacement at the peak and shear stiffness are summarised in Table 4.4. To compare with experimental results, the graph and values obtained from the laboratory tests were also described respectively. From these results, it was understood that the peak strength and residual strength increase with the increase of the confining pressure. This tendency is consistent well with the results obtained from laboratory tests.

Rock bolt with ribs

Simulations of the low-strength and high-strength mortars produced different results. During the laboratory tests, the mortar strength had a significant impact on both the crack distribution and load–displacement curve. Figure 4.12 shows the horizontal stress (σ_{xx}) and crack distributions, where the left side of the figure represents the results with the low-strength mortar, and the right side of the figure describes the results with the high-strength mortar. Figure 4.13 illustrates the load–displacement curves obtained from the DDA simulations (SS=300), with three important points (i), (ii), (iii) depicted. These three points correspond directly to the points (i), (ii), (iii) in Figure 4.12. It can be seen that point (i) is where the two load-displacement curves in Figure 4.13 started to diverge; point (ii) corresponds to the peak shear strength; and point (iii) is where horizontal displacement of rock bolt reached 2.5 mm

after the peak. As observed from the laboratory tests, when the low-strength mortar was employed, the horizontal cracks dominated. On the other hand, when the

Figure 4.9 Load-displacement curve obtained from sensitivity analyses

Figure 4.10 DDA results after the shearing simulation (Upper) and picture of the test piece which was taken after the shear test (Lower)

Confining pressure = 2.0MPa

Confining pressure = 4.0MPa

Confining pressure = 6.0MPa

Figure 4.11 Load – displacement curve obtained from numerical simulations(Left) and experiments (Right)

Table 4.4 Summary of DDA results without rib

	Peak Strength (MPa)	Residual Strength (MPa)	Displacement at peak (mm)	Shear Stiffness (MPa/mm)
Case 01 (DDA)	2.1	1.5	0.3	7.0
Case01-03 (Lab) AVE.	2.2	1.3	0.4	6.2
Case 02 (DDA)	4.4	3.3	0.5	9.1
Case04-06 (Lab) AVE.	3.5	2.2	0.6	5.7
Case 03 (DDA)	5.1	4.3	0.7	7.4
Case07-09 (Lab) AVE.	5.4	3.7	0.8	8.5

high-strength mortar was utilised, inclined cracks and horizontal cracks occurred and penetrated into the mortar material. As a result, the mortar failed brittly. For the DDA results, at point (i), the differences in the crack distribution between the low-strength mortar and the high-strength mortar can be observed from Figure 4.12 (a), while the horizontal stress was similarly concentrated in the area in front of the ribs. To be specific, the low angle cracks were initiated from the edge of ribs in the case of the low-strength mortar. Meanwhile, the inclined cracks were generated in the case of high-strength mortar.

The reason for the different crack patterns in this stage can be considered as follows. In the case of low-strength mortar, since the shear strength of the mortar was small, the mortar could not resist the horizontal shear force that was caused by the stress concentration in front of the rib. As a result, shear cracks formed horizontally at the rib height. On the other hand, in the case of high-strength mortar, though the stress distribution was similar to the result for low-strength mortar, the mortar

could resist the horizontal shear force because of its high shear strength. Therefore, the inclined cracks, which are considered to be tensile cracks around the tip of ribs, could be observed. As a result, tensile failure occurred as the mortar moved to the right with the ribs before the horizontal shear failure occurred. So it can be concluded that the crack distribution is highly dependent on the relationship between the shear strength and tensile strength of the bond material. After peak point (ii), significant differences in the crack propagation can be observed. Similar to the experimental results, when the low-strength mortar was utilised, horizontal cracks were dominant, while both inclined and horizontal cracks could be found when the high-strength mortar was employed (Figure 4.12 (b)). Moreover, the result for the low-strength mortar showed that the upper part of the mortar was separated from the lower part by the dominated horizontal cracks, whereby the horizontal stress in the upper part was not influenced significantly during the shear test. In contrast, the result with the high-strength mortar showed that the horizontal stress was affected even in the upper parts, therefore the larger parts of the mortar could be utilised for resisting the shear loading (Figure 4.12 (c)). From these results, it is concluded that these differences in the crack distribution can be considered to be one of the major reasons why the peak strength of the high-strength mortar is larger than that of the low-strength mortar. However, same as the experimental results, the damage in the mortar after the shearing test was significant compared to the result for the low-strength mortar. As shown in Figure 4.13, each load-displacement curve obtained from the DDA results was not exactly same

as that for laboratory tests. This is probably due to the limitation of 2D modelling and the effect of the mortar particle size (i.e., the particle size in the DDA model is much larger than the true mortar particle size). However, the DDA simulation results show at least similar trend in terms of the crack distribution and the load-displacement curve to the laboratory test results. Therefore, it can be concluded that the simplified DDA rock bolt model can be an useful tool for analysing the debonding process along the bolt-mortar interface and the crack propagation into the mortar in detail.

Figure 4.12 DDA results after shearing simulation

(Left: Low strength mortar, Right: High strength mortar)

Figure 4.13 Load-displacement curve (numerical simulation)

(RA=Rib angle, MS=Mortar strength, CP=Confining pressure)

Table 4.5 Summary of DDA results with rib

	Peak Strength (MPa)	Residual Strength (MPa)	Displacement at peak (mm)	Shear Stiffness (MPa/mm)
Case 04 (DDA)	2.3	1.9	0.6	3.8
Case 05 (Lab)	2.8	2.0	0.5	5.6
Case 05 (DDA)	3.4	2.0	0.8	4.3
Case 11 (Lab)	4.4	1.8	0.9	4.9

4.3. Effect of rock bolt configuration

In the previous section, it has been verified that DDA is an effective tool to simulate the interface behaviour of the simplified rock bolt model with/without ribs in terms of the crack distribution and load-displacement curve. In this section, the DDA based rock bolt model was used to study the effects of the main rock bolt parameters (i.e., rib angle, the ratio of the mortar thickness to the rib height, rib space and rib shape) on mortar-bolt interface behaviour.

4.3.1. Numerical simulation cases and results of basic cases

Table 4.6 presents a list of simulation cases with varying bolt profile configurations. The properties of the mortar are shown in Table 4.7, and the high strength mortar was used in this study. All parameters were determined based on the basic simplified model shown in Figure 4.14 (rib angle = 90° , the ratio of the mortar thickness to the rib height $\alpha = 12 \text{ mm} / 2 \text{ mm} = 6$, rib space = 17.8 mm, rib shape = square). As a first step of parameter studies, the crack distribution and the load-displacement curve of the basic case were compared with those of the laboratory test to verify the input parameters.

Figure 4.15 and Figure 4.16 show a comparison of the results from the DDA simulations and the laboratory experiments. At an early stage of the shear test (horizontal displacement of the rock bolt obtained from DDA=0.3mm), several inclined tensile and shear cracks from the tip of the ribs can be observed in the DDA simulation (Figure 4.15(a)). Similar oblique cracks were observed in the laboratory test, as shown in Figure

4.15(b). A larger inclined tensile crack was generated at the left rib when the horizontal displacement of the rock bolt was 0.6 mm (Figure 4.15(c)), which was probably due to the moving direction of the rock bolt. This oblique tensile crack was also seen during the laboratory test, as shown in Figure 4.15(d). After that, similar to the laboratory test results, DDA results showed that the inclined cracks were generated first, followed by the sub-horizontal shear cracks. These horizontal cracks propagated and coalesced gradually, and finally, the shear stress tended to be a constant (2.1 MPa) at the residual stage of the DDA simulation. As shown in the comparison study, the numerical simulation results of the basic case showed a good agreement with those of laboratory tests in terms of crack distribution and the trend of load-displacement curve, which verified and confirmed the capability of the established DDA model in the investigation of the mortar-bolt interface behaviour.

Table 4.6 List of DDA simulation cases and the configurations

Parameter	Rib angle		
	30°	60°	90°*
Image			
Parameter	Ratio of mortar thickness to rib height (α)		
	$\alpha=3.0$	$\alpha=6.0^*$	$\alpha=12$
Image			
Parameter	Rib space (1.0D = 17.8 mm)		
	0.5D	1.0D*	2.0D
Image			
Parameter	Rib shape		
	Square* (Corner radius = 0.0)	Half circular w/ flat surface (Corner radius = 1.0)	Half circular w/o flat surface (Corner radius = 2.0)
Image			

***Bold** : Basic case

Table 4.7 Input data for DDA simulation (Mortar)

Material	Parameter	Value
Mortar (High strength: UCS=30MPa)	Unit weight (kN/m ³)	23
	Young's modulus (GPa)	7.3
	Poisson's ratio	0.2
	Friction angle (°)	40 / 42.5
	(Initial / residual)	
	Cohesion (MPa)	8.5 / 3.0
	(Initial / residual)	
	Tensile strength (MPa)	1.8 / 1.0
	(Initial / residual)	

Figure 4.14 Simulation mesh for the basic model

(Rib angle = 90°, Ratio of the mortar thickness 12mm to the rib height

2mm = 6.0, Rib space = 17.8mm, Rib shape = square)

(a)DDA (Horizontal displacement
of rock bolt = 0.3 mm)

(b)Laboratory test
(Enlarged view)

At the beginning stage

(c)DDA (Horizontal displacement
of rock bolt = 0.6 mm)

(d)Laboratory test
(Overall view)

Before the peak shear strength

(e)DDA (Horizontal displacement
of rock bolt = 3.0 mm)

(f)Laboratory test
(Enlarged view)

After the peak shear strength

Figure 4.15 Comparison of the results from the DDA simulations (left)
and experimental tests (right)

Figure 4.16 Comparison of the load-displacement curves obtained from the DDA simulations (left) and experimental tests (right)

4.3.2. Results and discussion

Influence of rib angle

In addition to the basic case with 90° rib angle, more cases with a rib angle of 30° and 60° were simulated under a confining pressure of 0.5 MPa. Figure 4.17 shows the horizontal stress contour (σ_{xx}) and crack distribution obtained from the DDA simulations; and Figure 4.18 shows the corresponding load-displacement curves. Generally, the results with 60° rib angle were similar to those with 90° rib angle in terms of horizontal stress (σ_{xx}) and crack distribution (Figure 4.17). Shear strength and the corresponding horizontal displacement were also consistent with the results obtained from the basic case (Figure 4.18). The results with 30° rib

angle however showed a slight different trend. To be specific, a smaller number of cracks can be observed for the case with a rib angle of 30° , and the corresponding peak strength (0.9 MPa) was slightly lower than that in the other cases (1.1 MPa), which is probably due to the slip behaviour along the mortar-bolt interface arising from inadequate mechanical interlocking. From the displacement vector distribution (Figure 4.19), it can be seen that the observed slip behaviour occurred in the early stage (within a horizontal displacement of 1 mm). As a result, displacement vectors in the mortar have a trend in the upward direction, different from the other cases that have horizontal displacement vectors. In addition, the number of cracks was obviously less than that obtained from other cases. Therefore, it can be concluded that the bolt rib with an angle of 30° cannot resist significantly against the horizontal shear force because of the lack of mechanical interlocking at the boundary between the rib and the mortar, and the peak strength for 30° rib angle became smaller than those for 60° and 90° rib angles. In chapter 3, the author summarised that the effect of different rib angles was not significant under the experimental conditions. However, this slip behaviour was confirmed even in the laboratory tests as shown in Figure 4.20. When the rib angle was small (i.e., 20° or even 10°), it is reasonable to assume that the slip behaviour is likely to occur, and the corresponding results would be similar to the results of the rock bolts without ribs. Therefore, it can be summarised from these results that the rib angle should be larger (larger than 60°) in order to avoid the negative slip behaviour. Figure 4.21 shows the relationship between the rib angle and shear strength obtained from the

DDA simulations. The figure shows that when the rib angle was larger than 60° , the reinforcing effect of rock bolt remained similar. This finding is consistent with the laboratory tests by Tepfers (1979) and Kilic et al. (2003), in which they concluded that the rock bolt ribs with an angle between 45° and 105° produced similar effects; and the results with 60° rib angle were similar to those with 90° rib angle in terms of yield load, elastic displacement and failure load. Furthermore, when the confining pressure was increased to 2.0 MPa, the slip behaviour along the rock bolt and mortar was less likely to occur, whereby the effect of different rib angles slightly decreased as shown in Figure 4.22. These findings show a trend similar to that found in experiments in Chapter 3.

(a) At the beginning stage (Horizontal displacement = 0.5mm)

(b) Before the peak point (Horizontal displacement = 1.0mm)

(c) Around the peak point (Horizontal displacement = 1.5mm)

(d) After the peak point (Horizontal displacement = 3.0mm)

Figure 4.17 Horizontal stress and crack distribution

(Left: RA = 30°, Centre: 60°, Right: 90°)

Figure 4.18 Load-displacement curve with different rib angle

(a) At the beginning stage (Horizontal displacement = 0.5mm)

(b) Before the peak point (Horizontal displacement = 1.0mm)

(c) Around the peak point (Horizontal displacement = 1.5mm)

(d) After the peak point (Horizontal displacement = 3.0mm)

(e) Figure 4.19 Displacement vector

(Left: RA = 30°, Centre: RA = 60°, Right: RA = 90°)

Figure 4.20 Slip behaviour observed during laboratory tests

Figure 4.21 Relationship between rib angle and shear strength
(Confining pressure = 0.5 MPa)

Figure 4.22 Relationship between rib angle and shear strength
(Confining pressure = 2.0 MPa)

Influence of the ratio of mortar thickness to rib height α

In this section, the failure pattern of two other cases with different values of α is discussed, namely, $\alpha = 3$ (i.e., the mortar thickness is smaller than that in the basic case: 6 mm) and $\alpha = 12$ (i.e., the mortar thickness is larger than that in the basic case: 24 mm). Figure 4.23 shows the horizontal stress contour (σ_{xx}) and crack distribution obtained from the DDA simulations, Figure 4.24 shows the load-displacement curves. These results indicated that the model with a thicker mortar case ($\alpha = 12$) was similar to that obtained from the basic case ($\alpha = 6.0$) in terms of the crack distribution and peak shear strength of load-displacement curve. However, the inclined cracks, which were generated from the left rib, did not penetrate the whole mortar layer. The propagation of the inclined cracks terminated at approximately 18 mm from the mortar-bolt interface. Furthermore, the peak strength obtained from the thinner mortar case ($\alpha = 3$) was lower (0.9 MPa) than that obtained from other cases (1.1 MPa). As shown in Figure 4.23(a), several inclined tensile cracks penetrated the mortar at the initial stage (horizontal displacement of 0.5 mm). The horizontal stress was concentrated significantly in front of each rib for the case with smaller mortar thickness at this time, whereby the number of cracks observed could be more than that in the other two cases. As a result, the peak strength for the thinner mortar case ($\alpha = 3$) decreased. Figure 4.25 shows the relationship of the ratio of mortar thickness to rib height and shear strength obtained from the DDA simulations. An optimum ratio of mortar thickness to rib height of no less than 6.0 was recommended based on Figure 4.25, and the effect of rock bolting (in terms of shear

strength) remained similar even though a larger ratio α was used. Considering the soundness of the bond material, the ratio α was suggested to be approximately 9.0 because there were no cracks observed in the simulation above 18 mm from the mortar-bolt interface. In practice, as the cost for drilling and fixing materials increases with the increase in the borehole size, it is therefore suggested that the ratio α is less than 12, i.e. the mortar thickness is smaller than 24 mm.

(a) At the beginning stage (Horizontal displacement = 0.5mm)

(b) Before the peak point (Horizontal displacement = 1.0mm)

(c) Around the peak point (Horizontal displacement = 1.5mm)

(d) After the peak point (Horizontal displacement = 3.0mm)

Figure 4.23 Horizontal stress and crack distribution

(Left: $\alpha = 3.0$, Centre: $\alpha = 6.0$, Right: $\alpha = 12$)

Figure 4.24 Load-displacement curve with different mortar thickness

Figure 4.25 Relationship between α and shear strength

Influence of rib space

As described earlier, the rib space of the basic case was 17.8 mm (1.0D). In addition to this basic case, additional two cases were studied, i.e. a closer space ($0.5D = 8.9$ mm) and a longer space ($2.0D = 35.6$ mm). Figure 4.26 shows the comparison results of the horizontal stress contour (σ_{xx}) and crack distribution. Figure 4.27 presents the displacement vectors, and Figure 4.28 shows the load-displacement curves.

The stress in front of rib was higher for the case with a larger space (2.0D) than those for other cases; this is due to the fact that the number of ribs within a certain length (e.g., within 80mm in this simplified model) decreases as the rib space becomes larger. Hence, more cracks can be observed from the beginning stage of shear test (at a horizontal displacement of 0.5 mm). Second, the crack distribution was also different from that in the basic case. In the basic case (1.0D), horizontal shear cracks were developed from the tip of a rock bolt rib to a front rib, and finally, several cracks parallel to the boundary between the rock bolt and mortar were generated. These parallel cracks can be confirmed by laboratory tests, and this failure pattern is called the ‘parallel shear failure mode’ (Cao, 2012). On the other hand, as the interval space became large, horizontal shear cracks, which generated around the tip of a rock bolt, did not penetrate the mortar horizontally but penetrated towards the downward direction and finally reached the mortar-bolt interface. These cracks were also observed in laboratory tests, and this failure pattern is called ‘dilatational slip failure’ (Cao, 2012). From the numerical simulation results, the distance between the rib and the

point where the inclined cracks can reach was found to be approximately 25 mm (about 1.5D). The reason for the different crack distributions can be explained as follows. When the rib space was small (i.e., 0.5D and 1.0D), as shown in Figure 4.27, the displacement vectors in the mortar layer between the two ribs indicated that the mortar uniformly moved towards the shear direction. Therefore, the horizontal shear plane can be formed as if a direct shear test was carried out between the two ribs. On the contrary, when the rib space became larger (i.e. 1.5D), the displacement vectors in the mortar between two ribs were found to be not uniform. To be specific, the mortar in front of the rib moved forward significantly, but the mortar located far from the rib was not considerably affected. This vector distribution between the two ribs was similar to that when a uniaxial compression test was performed with the mortar specimen, whereby the inclined shear plane was observed.

The peak strength for a larger rib space was slightly lower (1.0 MPa) than that for the basic case (1.1 MPa). This is because the mortar failure patterns were different from those described above, and the stress was concentrated at the initial stage because of the small number of ribs; thus, the mortar cannot resist the higher shear loading. When the rib space was small (0.5D), the peak strength also became slightly lower (1.0 MPa) than that in the basic case (1.1 MPa). With regard to crack propagation, tensile cracks were generated initially from the tip of the left rib and further penetrated the mortar; this is similar to crack propagation observed in the basic case. Afterwards, more cracks generated from the tip of each rib, and they reached the front rib quickly due to the small rib

space. Finally, the cracks were connected, and the shear stress showed a residual value when parallel shear cracks coalesced completely. Similar results were observed by Aziz et al. (2008) who concluded that as the rib space became smaller, the bond material can be easily sheared horizontally, whereby the reinforcement effect of the rock bolt would be smaller. Figure 4.29 shows that the relationship between the rib space and peak shear strength obtained by the DDA simulations, which indicated that the optimum rib space should be between 1.0D and 1.5D (17.8 and 26.7 mm) although the difference in the peak strength was not significant. The recommended rib space by Aziz et al (2008) was 37.5 mm (2.1D), which was slightly larger than our recommendation, and the reason for the discrepancy might be due to the differences in the rib height and mortar strength. From our DDA simulation results, it was considered that the parallel shear failure mode did not occur if the rib space was larger than 25 mm (1.5D), in which case downward inclined cracks, which are called the dilatational slip failure mode, could be generated.

(a) At the beginning stage (Horizontal displacement = 0.5mm)

(b) Before the peak point (Horizontal displacement = 1.0mm)

(c) Around the peak point (Horizontal displacement = 1.5mm)

(d) After the peak point (Horizontal displacement = 3.0mm)

Figure 4.26 Horizontal stress and crack distribution

(Left: 0.5D, Centre: 1.0D, Right: 2.0D)

(a) At the beginning stage (Horizontal displacement = 0.5mm)

(b) Before the peak point (Horizontal displacement = 1.0mm)

(c) Around the peak point (Horizontal displacement = 1.5mm)

(d) After the peak point (Horizontal displacement = 3.0mm)

Figure 4.27 Displacement vector

(Left: 0.5D, Centre: 1.0D, Right: 2.0D)

Figure 4.28 Load-displacement curve with different rib space

Figure 4.29 Relationship between rib space and shear strength

Influence of rib shape

With regard to the rib shape, the basic case had a 2 mm square rib on one side (Corner radius = 0.0 mm). In addition to this basic case, two cases with circular ribs were simulated. The first case had a half circle with a radius of 1mm and a rectangle with a height of 1mm (Corner radius = 1.0 mm), and the second case had a half circle with a radius of 2 mm (Corner radius = 2.0 mm).

Figure 4.30 shows the horizontal stress and crack distribution, and Figure 4.31 shows the load-displacement curves. The load shown in displacement curves indicated that the peak strength for both cases (1.0 mm corner radius and 2.0 mm corner radius) became slightly lower (1.0 MPa) than that for the basic case (corner radius = 0.0 mm; 1.1 MPa). This could be due to the fact that when a circular rib was used, the circular shape made the horizontal stress in front of each rib less concentrated (but the influenced area was spread widely) as shown in Figure 4.30, and inclined cracks (with relatively high angles) were initiated and penetrated the mortar. This stress condition was similar to the result with a lower rib angle (i.e., 30°), but the slip behaviour did not occur because the rib shape did not have a flat surface. As a result, a number of steep and oblique cracks were observed in both cases. Since the inclined angle of cracks were steeper than the basic case, the mortar was split easily and became more difficult to resist against shearing, whereby both peak strengths were probably decreased. On the other hand, when the rib shape was square, though the stress was indeed concentrated around the sharp edge, horizontal cracks did not appear in the initial stage because a high-

strength mortar was used in these parametric studies. Therefore, the square shape is recommended for the three cases (see Figure 4.32). However, as shown in Chapter 3, when a low-strength mortar was used for the bond material, even at the initial stage, the mortar was easy to be sheared horizontally because of the stress concentration at the corner of square ribs. As a result, the relationship between the rib shape and peak shear strength might change for the three cases. Specifically, in the case with a lower strength mortar, a circular rib might be recommended because the stress concentration in front of each rib will be reduced. From this point of view, though the square rib can be recommended as described above, a square rib with slightly rounded corner (corner radius < 1.0 mm) might be considered as a preferable shape because this slightly rounded corner can reduce stress concentration and achieve a proper reinforcing effect even though various mortars were used for bond materials.

(a) At the beginning stage (Horizontal displacement = 0.5mm)

(b) Before the peak point (Horizontal displacement = 1.0mm)

(c) Around the peak point (Horizontal displacement = 1.5mm)

(d) After the peak point (Horizontal displacement = 3.0mm)

Figure 4.30 Horizontal stress and crack distribution

(Left: Square – Corner $r = 0.0\text{mm}$, Centre: Circular w/ flat surface – Corner $r = 1.0\text{mm}$, Right: Circular w/o flat surface – Corner $r = 2.0\text{mm}$)

Figure 4.31 Load-displacement curve with different rib shape

Figure 4.32 Relationship between rib shape and shear strength

4.3.3. Classification of induced cracks

One main advantage of using the DDA instead of continuum modelling is its capability to model multiple crack distribution during the shear tests, as continuum-based methods such as the FEM are not efficient to model multiple cracks. First, the failure modes obtained from DDA simulations are classified into four groups, as summarised in Table 4.8: tensile failure mode, combined failure mode, parallel shear failure mode and dilational slip failure mode.

Table 4.8 Classification for the mortar failure pattern

Mode	1.Tensile failure (large-angle crack)	2. Combined failure : Tensile and shear crack (relatively large-angle crack)
Image	A schematic diagram of a mortar joint between two ribs. A red line representing a crack is shown at a steep angle, labeled '80 ~ 90°'. The components are labeled 'Bolt', 'Rib', and 'Mortar'.	A schematic diagram of a mortar joint between two ribs. A red line representing a crack is shown at an angle, labeled '20 ~ 60°'. The components are labeled 'Bolt', 'Rib', and 'Mortar'.
Mode	3. Parallel shear failure (horizontal crack)	4. Dilational slip failure (inclined crack : downward)
Image	A schematic diagram of a mortar joint between two ribs. A red line representing a crack is shown at a shallow angle, labeled '0 ~ 10°'. The components are labeled 'Bolt', 'Rib', and 'Mortar'.	A schematic diagram of a mortar joint between two ribs. A red line representing a crack is shown at a downward angle, labeled 'About -20°'. The components are labeled 'Bolt', 'Rib', and 'Mortar'.

The tensile failure mode was observed at the beginning stage of shearing in all the simulation cases (horizontal displacement of rock bolt = 0.5 mm). The initial crack was generated at the tip of the rib located farther away from the loading point, and the angle of this crack was found to be very steep (around 80°). In fact, this steep tensile crack was confirmed in many laboratory tests conducted by the authors as shown in Figure 4.15(d), and when the mortar height was small (6.0 mm), this steep inclined crack easily penetrated the mortar, whereby the peak shear strength decreased dramatically. Thus, because this crack tended to split the mortar into two parts, this failure mode can be considered as an important failure pattern.

The combined failure mode comprised inclined tensile and shear cracks, which were generated from the tip of each rib. When compared with the tensile failure mode, the angle of these cracks was not large, ranged from 20° to 60° . In these parametric studies, because a high strength mortar was used, the inclined tensile cracks were the dominant ones at the beginning. After that, as the shear force increased, shear cracks also initiated, and the angle of these inclined cracks became small (close to 0°). This combined failure mode can be found in the laboratory tests conducted by Aydan (1989), in which many shear tests reported the existence of low angle combined tension-shear crack (LACTSC), consistent with DDA simulation results in this study. As a large area of mortar resisted against the shear force, this failure mode was recognised as one of the most important mode that strongly affected the peak shear strength. However, when the circular shape was used, this inclined crack

penetrated until close to the boundary between the mortar and rock and did not change its angle horizontally. In such case, these cracks separated the mortar into two blocks, as a result, the shear strength was reduced.

The parallel shear failure mode was characterised by horizontal cracks that connected one rib with other ribs. This type of crack can be seen in all cases, especially when the rib space was small. These cracks were generated almost horizontally, and the angle was from 0° to 10° . For a closer rib space ($0.5D$), the shear stress was close to the residual value as these parallel cracks connected each rib one by one. This failure mode was observed in a previous laboratory test by Jalalifer et al (2006). As summarised in Chapter 3: when a low-strength mortar was used, only this parallel shear failure was observed. Furthermore, as the mortar strength became higher, both inclined cracks and horizontal shear cracks dominated.

The dilational slip failure mode has downward diagonal cracks, which were generated from the top of a rib to the interface between the rock bolt and mortar. This failure mode occurred when the rib space was relatively large. The angle of these cracks was about -20° . The distance from the rock bolt rib to the point where these cracks reach was approximately 25 mm ($1.5D$). Even if the rib space is larger than this value, the dilational slip failure mode will be observed. This failure mode was also reported experimentally by Tepfers (1979) and with analytical solutions by Cao (2012); therefore, DDA results were found to be well consistent with past research. When the dilational slip failure mode occurred, it was found that the interface behaviour was similar to the case

with a low-angle rib (i.e., 30°), as a large rib angle can increase the peak shear strength. However, even with large-angle ribs, the reinforcing effect of the rock bolt can be decreased if the rib space was large enough to cause the dilational slip failure mode.

4.3.4. Implications of the study and recommendation for rock bolt configuration

Table 4.9 shows a recommendation for bolt configuration based on the results of this study.

In terms of the rib angle, a larger rib angle was preferable as it will not cause the negative slip behaviour. Therefore, the rib angle should be between 60° and 90° . Furthermore, when the confining pressure was increased, the slip behaviour along the rock bolt and mortar was less likely to occur. As a result, the effect of different rib angles becomes slightly lower though a large rib angle is still preferable.

With regard to the ratio of the mortar thickness to rib height, when the ratio α was greater than 6.0, the peak strength did not increase even though the mortar thickness was increased. However, from the view

Table 4.9 Recommendation for the rock bolt configuration

Rock bolt parameters	Recommendation
Rib angle	$60^\circ < \text{Rib angle} < 90^\circ$
Ratio of mortar thickness to rib height (α)	$9.0 < \alpha < 12$
Rib space (1.0D = 17.8 mm)	$1.0D < \text{Rib space} < 1.5D$
Rib shape	Square or slightly rounded corner (0.0mm < Corner radius < 1.0mm)

point of mortar soundness, the penetration of inclined tensile cracks into the mortar should be prevented completely. Therefore, it is recommended that the ratio α should be larger than 9.0 and smaller than 12 because the cost for drilling and bond materials can be more expensive as the mortar thickness increases.

With respect to the rib space, when the space is too small (i.e. $0.5D = 8.9 \text{ mm}$), the horizontal cracks, which were generated at the tip of a rib, could quickly reach the front rib, therefore, the reinforcing effect of the rock bolt was reduced. In addition, when the rib space was larger than $1.5D$ (26.7 mm), it is difficult to obtain a sufficient support from the rock bolt because of the occurrence of the dilational slip failure mode. From the relationship between the peak strength and rib space, a space of $1.0D$ to $1.5D$ is recommended as optimal.

Regarding the rib shape, it was found that the stress concentration around a rib can be partially avoided by using a circular rib head. However, the circular rib generated several inclined cracks with relatively large angles, whereby the mortar tended to be split into two blocks. As a result, the peak strength slightly decreased. These results implied that a square shape is recommended. However, when a low-strength mortar was used for the bond material during the laboratory tests, the mortar was sheared by the concentrated horizontal stress in front of each rib. Because the square shape generated more concentrated stress around its sharp corners, there might be some conditions in which the square rib worsens the effect of the rock bolt. In such case, a circular rib, which can avoid the stress concentration around the rib, might be a

better choice. These results indicate that a square rib or a square rib with slightly rounded corners (e.g. Corner radius <1.0 mm) is favourable.

The generated failure modes and recommended rock bolt configurations are summarised on the basis of the results of parametric studies with DDA simulations. Although these results are dependent on the strength of mortar and confining pressure, the classified failure modes with DDA simulation results are found to be well consistent with past experimental results.

4.4. Summary

In this Chapter, first, main results of shear tests were simulated to verify the accuracy of the DDA numerical simulation model by comparing the simulation results with laboratory tests. As the next step, by using the accurate DDA numerical simulation model, the effects of the main rock bolt parameters (i.e., rib angle, rib height, rib space and rib shape) on mortar-bolt interface behaviour were evaluated. The key findings obtained from the study are as follows:

From the verification of the DDA model,

- The simulated and experimental results for the rock bolt without ribs are in good agreement, which verifies the capability of the DDA in simulating the shear failure during pull-out tests.
- For the rock bolts with ribs, the crack distribution and load-displacement curves obtained from the DDA simulations were close to those observed from the laboratory tests. Therefore, it has been verified that the DDA

method can be an effective tool to simulate the crack initiation and propagation into the bond material.

From the parameter studies for the effect of the rock bolt profile,

- The angle of a rock bolt rib is suggested to be larger than 60° , which is because the slip behaviour is likely to occur in front of the rib surface as the rib angle becomes smaller, and consequently, the reinforcing effect is similar to the cases without ribs.

- The mortar thickness to rib height ratio should be more than 6.0 (mortar thickness 12 mm / rib height 2.0 mm) when the peak strength was the main concern. In addition, it should be greater than 9.0 (mortar thickness 18 mm/rib height 2.0 mm) when the mortar soundness was also considered. On the other hand, as the mortar thickness became small, the reinforcing effect of the rock bolt decreased because the cracks could penetrate the mortar easily even at the beginning stage of shearing.

- The preferable space between the rock bolt ribs was between 17.8 and 25.0 mm. In the case for rib spaces larger than 25 mm, the cracks, which were generated at the tip of each rib, did not develop horizontally but propagated downward and reached the interface between the rock bolt and mortar (dilatational slip failure mode).

- With regard to the rib shape, a circular rib can prevent the stress concentration around ribs; however, the large-angle inclined cracks were generated and penetrated the mortar. As a result of this behaviour, the mortar could be split easily, whereby the reinforcing effect became smaller than that in the case with square ribs. However, when square

ribs were employed, the stress was concentrated around the tip of each rib, therefore, horizontal shear cracks might dominate from the initial stage of the shearing if a low-strength mortar was used for the bond material.

Although the results obtained from parameter studies in this study were highly dependent on the mortar strength and the confining pressure, the findings in this research were well consistent with the previous results acquired by laboratory results. Furthermore, these results can be obtained by DDA that can well simulate the crack occurrence and propagation into the bond material. This research result, which considered the crack distribution, can contribute to well understanding the supporting mechanics of rock bolting especially where a quite large tunnel deformation occurs and the large displacement causes the many cracks inside the bond material.

As for the future works, it will be important to check not only the effect of rock bolt configurations (e.g., rib angle, rib height, rib space and rib shape) but also external factors (e.g., the influences of mortar strength and loading conditions) and combination of different parameters.

Chapter 5

Development of new rock bolt with DDA

rock bolt model

In this chapter, a new rock bolt, a type of the energy-absorbing rock bolt, was designed by using the DDA rock bolt model. Furthermore, prototypes of the rock bolt were prepared and laboratory pull-out tests were carried out.

Current energy-absorbing rock bolts in the deep mining have both the high loading capacity and the high deformation capacity. However, as described in Chapter 2, they permit the large displacement, but cannot control the ultimate displacement. In order to apply energy-absorbing rock bolt to civil engineering tunnels, a new energy-absorbing rock bolt has been developed. The new rock bolt is referred to as a deformation-controlled rock bolt (DC bolt). The DC bolt has the same characteristics of current energy-absorbing rock bolts (i.e., high loading and large deformation capacity) and also can control the rock displacement by the specially designed rock bolt anchor. As a result, it is possible to generate the tri-linear bond slip behaviour as shown in Figure 5.1, and it is also possible to control the amount of deformation considering the conditions (strength, fracture, etc.) and the stress of the surrounding ground. The developed rock bolt is mainly comprised of four components: 1) a smooth bar; 2) a threaded bar; 3) an end anchor whose diameter is larger than the

smooth bar; and 4) a ring which does not attach with the smooth bar as shown in Figure 5.2.

In this chapter, the supporting mechanism of the end anchor was determined via a detailed DDA modelling first, followed by the summary of the laboratory pull-out tests with prototype rock bolts. In the next chapter, a new DDA rock bolt element which can reproduce the appropriate tri-linear deformation behaviour was validated by comparing the DDA simulation results with laboratory results. Finally, a tunnel excavation example was simulated with the proposed energy-absorbing rock bolt element to evaluate the effectiveness of the newly-developed DC bolt.

Figure 5.1 Proposed tri-linear shaped load-displacement curve

Figure 5.2 Conceptual image of the deformation-controlled rock bolt

5.1. Simulation of the detailed anchor model with DDA

Figure 5.3 shows a simplified end anchor model of the proposed DC bolt. As described in the previous section, the DC bolt mainly consists of the rock bolt, the end anchor and the ring. In this model, shear force was applied at the right edge of the rock bolt horizontally. The rock model and mortar model were fixed at the right edge in order not to move horizontally. The ring model could not rotate during the simulations. Table 5.1 shows the properties of the rock bolt and the mortar material.

Figure 5.3 DDA model for the deformation-controlled rock bolt

Table 5.1 Input data for DDA simulations (anchor model)

Material	Parameter	Value	Material	Parameter	Value
Mortar	Unit weight	23 (kN/m ³)	Rock bolt	Unit weight	78 (kN/m ³)
	Young's modulus	7.3 (GPa)		Young's modulus	200 (GPa)
	Poisson's ratio	0.2		Poisson's ratio	0.3
	Friction angle	40/42.5 (°)		Friction angle	35/30 (°)
	(initial/residual)			(initial/residual)	
	Cohesion	8.5/3.0 (MPa)		Cohesion	(MPa)
	(initial/residual)			(initial/residual)	
	Tensile strength	1.8/1.0 (MPa)	bolt-mortar boundary		
	(initial/residual)				

Figure 5.4 shows the DDA simulation result of the horizontal stress and the crack distribution, and Figure 5.5 shows the load-displacement curve of the rock bolt obtained from the DDA simulation. Main results can be summarised as follows.

(a) At the beginning of shearing (horizontal displacement = 2.5 mm)

(b) During the ploughing behaviour (horizontal displacement = 10 mm)

(c) When the anchor hit the ring (horizontal displacement = 30 mm)

Figure 5.4 Simulation results of the horizontal stress and the crack distribution

Figure 5.5 Load-displacement curve obtained from the DDA simulation

At the beginning of the shear test (horizontal displacement = 2.5 mm), the horizontal stress was concentrated in front of the anchor, and the cracks were initiated from the anchor as shown in Figure 5.4(a). Even though the number of cracks was small, several cracks were generated from the interface between the threaded bar and the mortar. From this observation, it can be found that the anchor and the bolt-mortar boundary mainly generated the resistance force in the early stage corresponding the segment 9a) in Figure 5.5.

With a further increase of the pull-out force, the slope of the load-displacement curve gradually became smaller and finally showed a plateau (horizontal displacement = 5-30 mm). During this process, the anchor was found to move inside the mortar like a cone bolt, and many cracks were initiated and penetrated between the anchor and the ring as shown in Figure 5.4(b), which was named as deformable section in Figure 5.5.

Ultimately, when the anchor contacted with the ring, the stress concentration area was shifted to the mortar in front of the ring as shown in Figure 5.4(c). Furthermore, the load-displacement curve increased sharply again, as shown in Figure 5.5 (Secondary resistance). As a result, a newly designed rock bolt was found to show the ideal tri-linear behaviour as shown in Figure 5.1.

Next, in order to better understand the supporting mechanism of the deformation-controlled rock bolt, parameter studies were carried out with different bolt-mortar interface conditions. Figure 5.6 shows that the DDA simulation cases with different interface conditions. In these cases, Case 01(Basic case) was corresponding to the previous simulation case shown in Figure 5.3 and Table 5.1. On the other hand, the Interface property in Case 02 illustrates low friction (no bonding). Therefore, only anchor would resist against the pull-out force instead of the bolt-mortar interface. The interface property in Case 03 indicates lower friction than the basic case, but higher friction than Case 02. To be specific, the bolt-mortar interface property in front of the anchor point was set to low friction (no bonding) and interface behind the anchor point was high friction. Except the interface, the other properties and the shearing procedure were exactly same as the previous simulation.

Figure 5.7 illustrates the horizontal stress and crack distribution at the peak loading, and Figure 5.8 presents each load-displacement curve obtained from the DDA simulations. From these results, all curves show ideal tri-linear curves, similar to the basic model. Furthermore, the stress and crack propagation inside the mortar were also found to be similar to

those from the basic case. In other words, the stress was concentrated between the anchor and the ring at the beginning, followed by the increase of the cracks. After that, the area of stress concentration was shifted to the front of the ring when the anchor hit the ring. However, it is noteworthy that there were small differences in the peak load value at the end of primary resistance section. This means that the primary peak strength can be designed arbitrarily by adjusting the bolt-mortar interface properties. Meanwhile, the differences in the primary peak strength is not significant. Therefore, it is concluded that the existence of the anchor influences on the supporting effect more significantly than the interface properties between the bolt and the mortar.

Figure 5.6 DDA simulation cases with different interface properties

Figure 10 shows a schematic diagram of a concrete slab. The slab is supported by a red base labeled "Bolt". The left and right ends of the slab are labeled "No Cracks" in blue. The central area of the slab, where a crack is shown, is labeled "Many cracks" in red. A vertical color scale on the left indicates values from -40 to 0.07.

168

From the simulation result of this DDA-based anchor model, the following supporting mechanism of the new DC bolt can be concluded:

1. Primary resisting section:

The friction force between the threaded bar and grout and the end anchor start to resist against pull-out loading caused by the rock dilation (Figure 5.9(a)).

2. Deformable section:

The anchor itself starts to resist by ploughing inside the grout after the debonding occurs along the bolt-grout interface (Figure 5.9(b)).

3. Secondly resisting section:

When the anchor reaches the ring, the ring starts to resist against pull-out force (Figure 5.9(c)).

(a) Primary resistance along the bolt-mortar interface
(mainly anchor)

(b) Deformable section by the anchor ploughing inside the mortar

(c) Secondly resistance at the ring

Figure 5.9 Supporting mechanism of a deformation-controlled rock bolt

5.2. Laboratory pull-out tests with prototypes

A newly designed rock bolt was found to show the ideal tri-linear deformation behaviour in the previous subsection. As a next step, three m-long DC bolts were manufactured as shown in Figure 5.10, and laboratory pull-out tests were carried out. As explained earlier, the prototype consists of the smooth bar, the threaded bar, the anchor and the ring. The diameter of bar was 24 mm. The maximum and minimum diameters of the anchor were 31.7 mm and 30 mm, respectively, and the length of anchor was 90 mm. The diameter and the thickness of the ring were 44 mm and 6 mm, respectively. The distance between the anchor and the ring was defined as the allowing deformation “ δ ”.

Pull-out test cases and another specifications of rock bolts are listed in

Table 5.2. For comparison purposes, the result of a fully grouted rock bolt, which was pulled out in a construction site, was also used. Two DC bolts which have distance of 30 mm and 100 mm between the anchor and the ring were prepared. The boreholes were reproduced with steel pipes ($\phi = 50$ mm) and filled up with the mortar (UCS = approximately 10 MPa). After the appropriate mortar strength was confirmed, the pull-out force was loaded by the centre-hole jack. Figure 5.8 shows the photo of laboratory pull-out test setup. During the pull-out tests, the pull-out load and the displacement of rock bolt head were monitored by the load cell and two displacement metres attached at the edge of the rock bolt.

Detailed Profile

Photo of the anchor part

Figure 5.10 Prototype of deformation-controlled rock bolt

Table 5.2 Test cases and specifications of the rock bolt

Case	Bolt type	Bolt length (m)	Thread length (mm)	Allowing deformation (mm)	Mortar strength (MPa)
1	Fully grouted rock bolt		3000	----	10
2	Deformation-controlled rock	3	205	30	10
3	bolt		205	100	10

Figure 5.11 Photo of the laboratory pull-out test condition

Figure 5.12 Test specimen dismantled after removing the half-split pipe

Figure 5.12 shows one of the test specimen (case 3) dismantled after removing the half-split pipe. The observation result revealed the anchor moved inside the mortar and ultimately hit the ring for both cases (i.e., 30 mm or 100 mm between the anchor and the ring).

However, when the pull-out force reached around 200 kN, the ring was broken before the steel bar was ruptured. So a thicker ring can be recommended when the DC bolts are applied to the actual construction site.

Figure 5.13 shows the load-displacement curves obtained from the pull-out tests, for the fully grouted rock bolt and two DC bolts. In the case of fully grouted rock bolt, the pull-out force started climbing linearly until the pull-out force closed tensile strength of the steel bar (450 MPa), then the test was stopped to prevent the rock bolt from rupturing. At the 200 kN of pull-out loading, the displacement of rock bolt head was only 5 mm. In contrast, for the newly developed DC bolts, the tri-linear shaped load-displacement curves can be observed in both cases. To be specific, both curves increased linearly until reaching the 100 kN of pull-out loading (i.e., primary resisting section). At this point, the horizontal displacement was approximately 10 mm, and therefore the shear stiffness was slightly smaller than that of the fully grouted rock bolt. This is because that the DC bolt resisted the pull-out force only at the far end of the rock bolt, whereby the extension of the smooth bar between the collar and the anchor of the rock bolt were induced. After the pull-out loading reached 100 kN, the load-displacement curves for both DC bolts showed transition points and flattened out (deformable section), and load-displacement

curves showed the second transition points at approximately 30 mm and 100 mm of the horizontal displacement, respectively. Afterwards, the two curves gradually increased again, indicating that both DC bolts restarted to resist against the pull-out force (secondly resisting section). The slope of both load-displacement curves after second transition points were found to be almost equal. When the rings were broken at the 200 kN of the pull-out force, the displacement at the rock bolt head was 60 mm and 135 mm, respectively. Since the displacement of the fully grouted rock bolt was 5 mm, the advantage of the newly developed deformation-controlled rock bolt against the large tunnel displacement could be verified. Obtained results in this section show a good agreement with the assumed results based on the distance between the ring and anchor. However, it is recommended to evaluate the repeatability of test results since these pull-out tests were carried out only once for each case.

Figure 5.13 Load-displacement curves obtained from laboratory pull-out tests

5.3. Bolt installation test

In previous section, the supporting mechanism and the main features of the DC bolt were obtained by the DDA simulations and laboratory pull-out tests with the prototypes. However, because of its complicated bolt profile, it might be difficult to install the DC bolt in construction sites efficiently. The mortar might not be filled up around the anchor and the ring properly, whereas it is easy to install the fully grouted rock bolts into a borehole filled up with a fresh mortar. As the DC bolt profile is more complex, the bolt installation becomes more difficult because of the additional resistance. Furthermore, there might be a void (i.e., the area where the mortar does not fill up) in the narrow space (i.e., in the vicinity of the anchor and the ring), whereby the reinforcement effect will be decreased. Therefore, laboratory bolt installation tests were carried out with transparent acrylic pipes and DC rock bolts (Figure 5.14).

Figure 5.14 Acrylic pipes (left) and deformation-controlled rock bolts (right)

The procedure of bolt installation strictly followed the actual procedure at tunnel construction sites. As shown in Figure 5.15, the acrylic pipes, which simulate boreholes, were filled up with the mortar, and after that, DC bolts were installed in the pipes. Totally, three DC bolts were installed during the test.

Figure 5.15 Acrylic pipes filled up with mortar (left) and bolt installation (right)

The obtained results can be summarised as follows;

- The average installation time was approximately 15 sec, and three DC bolts were installed by manually as shown in Figure 5.15. These results were found to be almost equivalent to the installation with widely used fully grouted rock bolts. Therefore, DC bolts can be used in the construction sites without any installation problems.
- Figure 5.16 shows one of the observation results of the mortar filling condition after removing the half-split acrylic pipes. From this result, it can be confirmed that the mortar was filled up surrounding the rock bolt including the especially narrow space (i.e., the vicinity of the

anchor and the ring). Therefore, the sufficient reinforcing effect can be obtained even in the actual construction sites.

Figure 5.16 Mortar filling condition between the anchor and the ring (upper) and behind the anchor (lower)

5.4. Summary

In this chapter, a new type of energy-absorbing rock bolt, that is referred to a deformation-controlled rock bolt (DC bolt), was proposed. Its reinforcing mechanism was determined by the DDA simulation with the simplified rock bolt model. This DDA simulation model had been validated by comparing with the laboratory shear tests in Chapter 4. After that, in order to check whether the ideal tri-linear bond-slip behaviour can be obtained, pull-out tests were carried out using the prototypes. Finally, rock bolt installation tests were carried out to understand its workability and the mortar filling condition.

The obtained results were summarised as follows;

- From the results of DDA simulation, it could be found that the newly proposed DC bolt shows the ideal tri-linear shaped load-displacement curve. Furthermore, by observing the stress condition and crack distribution inside the mortar, its supporting effect can be classified into the following three stages;

1. Primary resisting section:

The friction force between the steel bar and grout and the end anchor start to resist against pull-out loading caused by the rock dilation.

2. Deformable section:

The anchor itself starts to resist by ploughing inside the grout after the debonding occurs along the bolt-mortar interface.

3. Secondly resisting section:

When the anchor hits the ring, the ring starts to resist against pull- out force.

- From the results of pull-out test using the prototypes, it can be seen clearly that the DC bolt could withstand large displacement compared to the widely used fully grouted rock bolt. Furthermore, the optimum DC bolt for various geological conditions can be designed arbitrarily by adjusting the allowing deformation, which is defined as the distance between the anchor and the ring.
- From the results of bolt installation tests that reproduced an actual installation procedure in tunnel construction sites, the DC bolt can be installed with same time for installation as the fully grouted rock bolt. Furthermore, it can be found that the mortar was filled up well even in the narrow space (i.e., the vicinity of the anchor and the ring) by the observation after the mortar hardening.

Chapter 6

Evaluation of the supporting effect of the new DC bolt

6.1. Brief introduction of DDA rock bolt element

In this chapter, the DDA was employed to evaluate the supporting mechanism of a specially designed anchor and the reinforcement effects of a DC bolt by simulating the laboratory pull-out test and the tunnel excavation. In Chapter 4, the DDA was verified as the useful tool to simulate the crack distribution and to estimate the trend of the load-displacement curve by calibrating the DDA results with laboratory test results. In the previous chapter, the DDA shear test model was used for evaluating the supporting mechanism of the specially designed anchor in the DC bolt.

In the development of rock bolt element by DDA, Nie et al. (2014) firstly developed the rock bolt element in DDA, and it was further enhanced by Ma et al. (2016) and Nie et al. (2018) by introducing the tri-linear shear bond-slip model. A brief introduction of rock bolt theory adapted to the current DDA code is presented in the following.

The mathematical algorithm for a rock bolt element was developed by Hyett et al. (1996). When the surrounding rock dilates, the force equilibrium along the longitudinal direction of a rock bolt element can be written as:

$$\tau dx = -A_b d\sigma_{bx} \quad (6.1)$$

where τ is the unit shear force provided by the bond material, A_b is the area of the cross section of the rock bolt, σ_{bx} is the axial stress in rock bolt.

The stress-strain relationship of a rock bolt can be considered as linear elastic, and can be expressed as:

$$\sigma_{bx} = E_b \varepsilon_b = E_b \frac{dU_{bx}}{dx} \quad (6.2)$$

where E_b is the Young's modulus of a rock bolt and U_{bx} is the nodal displacement of the rock bolt. Submitting Eq. (6.2) into (6.1) gives:

$$\frac{d^2 U_{bx}}{dx^2} = \frac{\tau}{A_b E_b} \quad (6.3)$$

The unit shear force τ can be assumed as a linear function of the relative slippage, i.e.

$$\tau = k(U_{rx} - U_{bx}) \quad (6.4)$$

where k is the bond stiffness of the interface between the rock and rock bolt, U_{rx} is rock displacement. Submitting Eq. (6.4) into Eq. (6.3) gives:

$$\frac{d^2 U_{bx}}{dx^2} - \frac{k}{A_b E_b} U_{bx} = -\frac{k}{A_b E_b} U_{rx} \quad (6.5)$$

Assuming that the displacement varies quadratically, Eq. (6.5) can be rewritten as:

$$\frac{A_b E_b}{x^i - x^{i-1}} (U_{bx}^i - U_{bx}^{i-1}) + \frac{A_b E_b}{x^{i+1} - x^i} (U_{bx}^i - U_{bx}^{i+1}) = k \left(\frac{x^{i+1} - x^{i-1}}{2} \right) (U_{rx}^i - U_{bx}^i) \quad (6.6)$$

where x_i , x_{i-1} and x_{i+1} are the location at point i , $i-1$ and $i+1$, U_{bx}^i , U_{bx}^{i-1} and U_{bx}^{i+1} are the displacement of rock bolt at point i , $i-1$ and $i+1$, U_{rx}^i is the rock displacement at point i .

In Eq. (6.6), the terms on the left-hand side correspond to axial load at point i , while the terms on the right-hand side correspond to shear load at point i due to the relative movement between the rock and rock bolt. This equation indicates how a rock bolt approaches equilibrium when the surrounding rock dilates. In this model, rock bolt was considered as the simplified model, and therefore the differences of bolt dimension were not represented.

6.2. Simulation of pull-out test with DDA based DC element

The behaviour of the DC bolt was modelled by incorporating a unique tri-linear bond-slip model under the framework of the DDA model as shown in Figure 6.1.

Figure 6.1 A tri-linear bond-slip model with DDA

The proposed tri-linear bond-slip model can be expressed by:

$$\tau = k \cdot s + c \quad (6.7)$$

where τ is the shear stress at the bolt-rock interface; s is the relative displacement between the bolt and the rock.

The shear force on the bolt-rock interface F_s for a unit length can be expressed by:

$$F_s = \pi \cdot D \cdot \tau = \pi \cdot D \cdot k \cdot s + \pi \cdot D \cdot c \quad (6.8)$$

The parameters k and c are defined for each of the three stages (I, II, III as shown in Figure 6.1) as follows:

$$\text{When } 0 \leq s \leq s_1: \quad k = k_1 = \frac{\tau_1}{s_1}; \quad c = c_1 = 0 \quad (6.9)$$

$$\text{When } s_1 \leq s \leq s_2: \quad k = k_2 = \frac{\tau_2 - \tau_1}{s_2 - s_1}; \quad c = c_2 = \frac{\tau_1 \cdot s_2 - \tau_2 \cdot s_1}{s_2 - s_1} \quad (6.10)$$

$$\text{When } s > s_2: \quad k = k_3 = \frac{\tau_3 - \tau_2}{s_3 - s_2}; \quad c = c_3 = \frac{\tau_2 \cdot s_3 - \tau_3 \cdot s_2}{s_3 - s_2} \quad (6.11)$$

Figure 6.2 shows the DDA simulation model of the pull-out test with 3 m long rock bolt. The pull-out force was loaded on the head of the rock bolt with a loading rate of 10 kN/s. For comparison purposes, both the DC bolt and the fully grouted rock bolt were simulated as shown in Figure 6.3, and the corresponding simulation cases and the model parameters are listed in Table 6.1 and Table 6.2. For the DC bolt, two different deformation allowances were considered, i.e. 30 mm and 100 mm. The tensile strength of rock bolts was set to 450 MPa.

Figure 6.2 DDA simulation model of the pull-out test

Fully grouted rock bolt

Deformation-controlled rock bolt

Figure 6.3 DDA rock bolt model for the pull-out test simulation

Table 6.1 DDA simulation cases (pull-out test model)

Case	Bolt type	Bolt length	Allowable displacement (mm)
1	Fully grouted rock bolt		----
2	Deformation-controlled rock bolt	3m	30
3			100

Table 6.2 Rock bolt specifications (pull-out test model)

Item	Parameters	Value	Item	Parameters	Value
Bolt	Diameter (mm)	30	Bolt	Yield strength (MPa)	450
	Young's modulus (GPa)	200		Extreme strain (%)	24
Fixed node	k1(Pa/m)	4.0e9	Special node	k1(Pa/m)	3.8e8
	k2(Pa/m)	-4.0e9		k2(Pa/m) for Case02	1.7e6
				k2(Pa/m) for Case03	5.2e5
	k3(Pa/m)	1.0		k3(Pa/m)	3.8e8
	τ 1(kN/m)	1.0e6		τ 1(kN/m)	0.56e6
	τ 2(kN/m)	0.5e6		τ 2(kN/m)	0.61e6

The shear force and the axial force distribution during pull-out simulations are shown in Figure 6.4 and Figure 6.5, and the load-displacement curves are summarised in Figure 6.6. In the case of fully grouted rock bolt, the shear force was generated from the collar of the rock bolt, and it became smaller with the increase of the distance from the loading point. The axial force distribution showed similar trends as the shear force distribution, namely the maximum axial force was observed at the collar of the rock bolt, and it reduced with the increase of the rock bolt length. These results are quite consistent with typical pull-out test results with the fully grouted rock bolt (Farmer 1975). After the pull-out loading reached 270 kN, the shear force around the collar of the rock bolt started to decrease, which is probably due to the debonding failure along the bolt-rock interface. In the case of the DC bolt, the shear force was generated only at the far end of the rock bolt, and the shear force distribution was found to be entirely different from that of the fully grouted rock bolt. The axial force distribution was also found to be different from that of the fully grouted rock bolt, but it was confirmed to follow the typical distribution of the end anchor rock bolt.

(a) Fully grouted rock bolt

(b) DC bolt ($\delta = 30$ mm)

(c) DC bolt ($\delta = 100$ mm)

Figure 6.4 Shear force distribution during DDA pull-out simulations

(a) Fully grouted rock bolt

(b) DC bolt ($\delta = 30 \text{ mm}$)

(c) DC bolt ($\delta = 100 \text{ mm}$)

Figure 6.5 Axial force distribution during DDA pull-out simulations

Figure 6.6 Load-displacement curves obtained from DDA pull-out simulations

As shown in Figure 6.6, the simulated load-displacement curves clearly show the different characteristic of each rock bolt. The load-displacement curve of the fully grouted rock bolt shows a linear relationship between the pull-out force and the displacement of rock bolt until the rock bolt failed due to tension. The stiffness (i.e., the gradient of the graph) of the fully grouted rock bolt was higher than those of the DC bolts. So it can be concluded that the fully grouted rock bolt can be a suitable support element when the tunnel displacement has to be constrained. Meanwhile, it should be noted that a tensile failure may occur when a large deformation occurs in the fully grouted rock bolts.

In contrast, both load-displacement curves of the DC bolt show the tri-linear relationship between the pull-out force and the displacement at the rock bolt head. The pull-out force increased linearly until the pull-out force reached first transition point. After that, both curves showed a

plateau. After the second transition point (i.e., 30 mm or 100 mm), the curves started to rise sharply again. It is noted that the DC bolt can sustain a large deformation without failure, so it can be a very effective support under the squeezing conditions and it can be used for various types of displacement control by adjusting the amount of the deformation allowance.

The results obtained from the DDA pull-out simulations (Figure 6.6) are found to be consistent with the results obtained from the laboratory pull-out tests (Figure 5.13). It means that the applicability of the new DC bolt element could be verified by comparing the result of numerical simulations with laboratory tests. The DDA based rock bolt element enables us to utilise the new rock bolt element for the simulation of tunnel excavation, whereby the DC bolt can be designed to suit various geological conditions (i.e., the amount of deformation allowance; the number of rock bolt).

6.3. Tunnel excavation example simulated by DDA based DC bolts

Figure 6.7 shows the simulation model for the tunnel excavation. The tunnel was excavated in the rock mass which has two joint sets. The tunnel cover was 1.5 D, where D is the tunnel width ($D = 20$ m). The in-situ stress in the vertical and horizontal direction were set to 2.4 MPa and 4.8 MPa, respectively. The rock mass properties are listed in Table 6.3. Parameters of rock bolts are the same as those used in the simulation of pull-out tests, and the deformation allowance of the DC bolts was set to 100 mm. In order to evaluate the effect of rock bolts only, the steel set and

the sprayed concrete were not employed in the simulation. After simulating the tunnel excavation, the shear stress distribution, the axial force distribution of each rock bolt and the rock block displacement were evaluated.

Figure 6.7 DDA simulation model for the tunnel excavation

Table 6.3 Rock mass properties

Item	Properties	Value	Item	Properties	Value	Item	Properties	Value
Rock	Density (kg/m ³)	2600	Joint	Friction angle(°)	30	Joint	Cohesion (MPa)	2
	Poisson ratio	0.2	Joint set	Dip angle (°)	60	Joint set	Dip angle (°)	-30
	Elastic modulus (GPa)	50		Spacing(m)	8		Spacing(m)	4
			1			2		

Figure 6.8 and Figure 6.9 show simulation results with fully grouted rock bolts and DC bolts, respectively. The principal stress, the vertical stress distribution and shear force distribution along the rock bolt are illustrated in Figure 6.8. Furthermore, the axial force distribution along the rock bolt are displayed in Figure 6.9. For the fully grouted rock bolts, three out of five rock bolts were ruptured during the excavation due to the tension failure. For the other two rock bolts, the shear force was generated at the location of each discontinuity.

In contrast, the DC bolts were not ruptured, and they could withstand the large deformation due to the tunnel excavation. Furthermore, the observed shear force and the axial force distribution were consistent with those of pull-out test simulations. In other words, the shear force was generated only at the far end of the rock bolt, and the axial force was distributed uniformly along the rock bolt. From the vertical stress distribution, the excavation damaged zone with full grouted rock bolts could be seen slightly wider than that with DC bolts, because of the rupture of three rock bolts during the excavation.

Figure 6.10 shows the monitoring locations and the rock displacement after the excavation, respectively. As shown in Figure 6.10(a), monitoring locations from ① to ③ corresponds to the location where the ruptured fully grouted rock bolts were installed. The tunnel surface displacement with DC bolts was smaller than that with the full grouted rock bolts. This result demonstrated the advantage of the newly developed DC bolt under the severe condition where the large tunnel displacement occurs. On the other hand, for monitoring points ④ and ⑤

where no rock bolts were ruptured, the rock displacement with fully grouted rock bolts were smaller than that with DC bolts. This result implies that using the energy-absorbing rock bolts may allow the rock to move excessively, especially where the fully grouted rock bolt can withstand the rock deformation.

Figure 6.8 Shear force distribution after the excavation

Figure 6.9 Axial force distribution after the excavation

(a) Monitoring locations

(b) Rock block displacement after the excavation

Figure 6.10 Monitoring result of the rock displacement

6.4. Summary

In this chapter, in order to evaluate the supporting effect of a newly-developed DC bolt, the new DDA rock bolt element which can reproduce the results of pull-out tests (unique tri-linear shaped load-displacement curve) was developed.

In the DDA based rock bolt pull-out simulations, the distinctive behaviours in the shear force and axial force distribution of the fully grouted rock bolt and the DC-bolts were obtained, and the load-displacement curves were found to be well consistent with the result of laboratory pull-out tests.

The new DDA rock bolt element were applied to the DDA based tunnel excavation simulations. Several fully grouted rock bolts were ruptured due to the large tunnel deformation, resulting in the larger displacement and broader extent of the yielding zone. In contrast, no DC-bolts were ruptured during the excavation, whereby the tunnel displacement could be restricted based on the allowable displacements.

Chapter 7

Conclusion and recommendations

7.1. Conclusion

Rock bolts are essential tunnel support materials for the tunnel/cavern construction. Therefore, there have been a lot of research achievements since 1970s. As the interface behaviour between the rock bolt and the bond material is one of the crucial factors affecting the reinforcing effect of the rock bolt, this research aims to enrich the understanding in the interface behaviour and debonding mechanism.

Key findings based on shear tests and DDA modelling are summarised as follows.

Shear test

- For the rock bolt without ribs, no cracks were observed inside the mortar, and the shear behaviour was controlled only by the interface between the rock bolt and the mortar.
- For the rock bolt with ribs, rib angles did not significantly influence the load-displacement curve and crack propagation under specific testing conditions (2.0 MPa of confining pressure).
- When the low-strength mortar was used, horizontal cracks were mainly observed. In contrary, as the mortar strength increased, inclined cracks were generated and the mortar failed brittlely.
- As the confining pressure increased, the peak strength and residual strength also became high, regardless of whether or not there were ribs.

DDA modelling

- The simulation and experimental results for the rock bolt without ribs are in good agreement, which verifies the capability of the DDA in simulating the shear failure during pull-out tests. In order to obtain accurate results, selecting the proper spring stiffness value (both normal and horizontal stiffness value) was essential.
- For the rock bolts with ribs, the crack distribution and load-displacement curves obtained from the DDA simulations were close to those observed from the laboratory tests. Therefore, it has been verified that the DDA method can be an effective tool to simulate the crack initiation and propagation into the bond material.
- The angle of a rock bolt rib is suggested to be larger than 60° , which is because the slip behaviour is likely to occur in front of the rib surface as the rib angle becomes smaller, and consequently, the load capacity is similar to the cases without ribs when the confining pressure is relatively low.
- The mortar thickness to rib height ratio should be more than 6.0 (mortar thickness 12 mm / rib height 2.0 mm) when the peak strength was the main concern. In addition, it should be greater than 9.0 (mortar thickness 18 mm/rib height 2.0 mm) when the mortar soundness was also considered. On the other hand, as the mortar thickness became small, the reinforcing effect of the rock bolt decreased because the cracks could penetrate the mortar easily even at the beginning stage of shear test.

- The preferable space between the rock bolt ribs was between 17.8 and 25.0 mm.
- With regard to the rib shape, a circular rib can prevent the stress concentration around ribs; however, the large-angle inclined cracks were generated and penetrated the mortar. As a result of this behaviour, the mortar could be split easily, whereby the load capacity became slightly smaller than in the case with square ribs.

New energy-absorbing rock

- The anchor of the new energy-absorbing rock bolt, DC bolt, was designed based on the DDA detailed anchor model. After that, the performance of the DC bolt was examined by laboratory tests and numerical modelling. Key findings from these studies are summarised as follows.
- Supporting mechanism of the DC bolt can be classified into three stages.
  - 1) Primary resisting stage: the friction force between the threaded bar and grout and the end anchor start to resist against the pull-out loading caused by the rock dilation.
  - 2) Deformable stage: the anchor itself starts to resist by ploughing inside the grout after the debonding failure along the bolt-grout interface.
  - 3) Secondly resisting stage: when the anchor reaches the ring, the ring starts to resist against the pull-out force.
- From the laboratory pull-out tests, the newly-developed DC bolt could cope with much larger displacement than the fully grouted rock bolt.

Moreover, ideal tri-linear shaped load-displacement curve can be obtained depending on the distance between the anchor and the ring.

- In the DDA based rock bolt pull-out simulations, the distinctive behaviours in the shear force and axial force distribution of the fully grouted rock bolt and the DC-bolts were obtained, and the load-displacement curves were found to be well consistent with the result of laboratory pull-out tests.
- In the example of the DDA based tunnel excavation simulations, several fully grouted rock bolts were ruptured due to the large tunnel deformation, resulting in the larger displacement and broader extent of the yielding zone. In contrast, no DC-bolts were ruptured during the excavation, whereby the tunnel displacement could be restricted based on the allowable displacements.

As described above, the author has focused on the interface behaviour between the rock bolt and the bond material in the first part of this study. By comparing with the laboratory shear test results, the DDA-based numerical simulation model for the rock bolt interface behaviour had been developed. As the application of this numerical simulation model, the effect of rock bolt configurations on the supporting effect of rock bolt was analysed. Furthermore, the new type of energy-absorbing rock bolt was designed and its reinforcing mechanics was investigated by the DDA-based simulation model. The results obtained from this study enable readers to better understand the supporting mechanism and reinforcing effect of rock bolts.

7.2. Recommendations

Finally, the limitation of the current work and the recommendation for a future work are summarised as follows.

- Rock bolts are subjected to not only the axial force but also lateral force. In this study, only the axial pulling load due to the shear behaviour along the interface between the rock bolt and the bond material was considered. The lateral behaviour should be taken into account in the future work.
- In order to focus on the crack distribution in the bond material, shear tests were carried out with 2D-simplified rock bolt model and 2D-DDA was also employed to build the accurate rock bolt model. However, actual rock bolts show 3D-behaviour, therefore results obtained from 3D pull-out tests should be added to the current work. Furthermore, it probably be hard to observe the detailed interface behaviour between the bolt and the bond material with the pull-out test, 3D numerical modelling would be more meaningful.
- The newly-developed DC bolt has not been applied to actual construction sites yet. In order to improve the applicability of new rock bolt, application results must be required in a future work. Furthermore, the DC bolt was developed for the squeezing ground where the tunnel deforms slowly. In order to investigate the reinforcement effect for the dynamic conditions (i.e. rock burst or

earthquake), the additional tests such as the drop test should be carried out.

References

Arno, T., Patrick, S., and Markus, S. (2006) “Tunnelling in Squeezing Rock-Yielding Elements and Face Control”, Proc. of 8th International Conference on Tunnel Constructions and Underground Structures, Slovenia.

Asahina, D., Aoyagi, K., Kim, K., Birkholzer, J. T., Bolander, J. E. (2017) “Elastically-homogeneous lattice models of damage in geomaterials”, Computers and Geotechnics, 81, pp. 195–206.

Aydan, Ö., Ichikawa, Y. and Kawamoto, T. (1985) “Load bearing capacity and stress distribution in/along rockbolts with inelastic behavior of interface”, Proc. of 5th International Conference on Numerical Method in Geo-mechanics, pp. 1281-1292.

Aydan, Ö. (1989) “The Stabilisation of Rock Engineering Structures by Rockbolts”, Ph.D Thesis, Nagoya University.

Aydan, Ö., Akagi, T. and Kawamoto, T. (1993) “The Squeezing Potential of Rocks Around Tunnel”, Rock Mechanics and Rock Engineering, 26(2), pp. 137-163.

Aydan, Ö., Ebisu, S., Komura, S. and Kawamoto, T. (1995) “A unified design method for rock anchor foundations of super-high pylons”, Rock Foundation, pp. 279-284.

Aziz, N., Jalalifar, H., Remennikov, A. M., Sinclair, S. and Green, A. (2008) “Optimisation of the bolt profile configuration for load transfer enhancement”, Coal Operators' Conference, pp. 125-131.

Barla, G. (1995) “Squeezing rocks in tunnels” ISRM News Journal 2 (3&4), pp. 44-49.

Barla, G. (2001) “Tunnelling under squeezing rock conditions” Eurosummer-School in Tunnel mechanics, Innsbruck.

Bolander, J. E., Saito, S. (1998) “Fracture analysis using spring networks with random geometry”, Engineering Fracture Mechanics, 61, pp. 569–91.

Cao, C. (2012) “Bolt profile configuration and load transfer capacity optimisation”, Ph.D thesis, University of Wollongong.

Cundall, P. A. (1971) “A computer model for simulating progressive, large-scale movements in blocky rock systems”, Proc. of International Symposium on Rock Fracture, pp. 11-18.

Fabbri, D. (2004) “The Gotthard Base Tunnel: Project overview”, The 6th annual Tunnelling Conference.

DSI (2017) “Ground Control Solutions for Tunneling and Mining”, Product catalogue, pp. 109-111.

Farmer, I. W. (1975) “Stress Distribution along a Resin Grouted anchors”, International Journal of Mining Science and Technology, 12, pp. 347-351.

Friess, M. (2012) “Scratching the Surface? The use of surface scanning in physical and paleoanthropology”, Journal of Anthropological Sciences, 90, pp. 7-31.

Geo fronte (2009) “Present technology and development of rock bolting” (in Japanese).

Goodman, R. E. (1989) “Introduction to rock mechanics”, New York, Wiley, c1989. 2nd ed.

Goodman, R. E. and Shi, G.H. (1985) “Block Theory and Its Application to Rock Engineering”, Prentice-Hall International series in Civil Engineering and Engineering Mechanics.

Goodman, R. E., Taylor, R. L. and Brekke, T. L. (1968) "A Model for the Mechanics of Jointed Rock", Journal of the Soil Mechanics and Foundations Division, 94(3), pp. 637-660.

Goto, Y. (1971) "Cracks Formed in Concrete Around Deformed Tension Bars", Journal Proceedings, 68(4), pp. 244-251.

He, L., An, X. M., Zhao, X. B., Zhao, Z. Y. and Zhao, J. (2018) "Development of a Unified Rock bolt Model in Discontinuous Deformation Analysis", Rock Mechanics and Rock Engineering, 51(3), pp. 827-847.

Hoek, E. (2006) "Practical Rock Engineering", Rockscience, Hoek's Corner.

Hyett, A. J., Bawden, W. F. and Reichert, R. D. (1992) "The effect of Rock Mass Confinement on the Bond Strength of Fully Grouted Cable Bolts", International Journal of Rock Mechanics and Mining Science, 29(5), pp. 503-524.

Hyett, A. J., Moosavi, M. and Bawden, W. F. (1996) "Load distribution along fully grouted bolts, with emphasis on cable bolt reinforcement" International journal for numerical and analytical methods in geomechanics, 20, pp. 517-544.

Jager, A. J. (1992) "Two new support units for the control of rockburst damage", Proc. of International Symposium on Rock Support in Mining and Underground Construction, pp. 621-631.

Jalalifer, H., Aziz, N. H. and Hadi, M. (2006) "An Assessment of Load Transfer Mechanism Using the Instrumented Bolts", Coal Operators' Conference, University of Wollongong & the Australasian Institute of Mining and Metallurgy : pp. 255-265. <http://ro.uow.edu.au/coal/53>

Japan Society of Civil Engineers (2000) "Japanese standard for mountain tunneling".

Japan Society of Civil Engineers (2010) "Standard specifications for concrete structures - 2007 "Design". Japan", Japan Society of Civil Engineers.

Jean, D. H. and Mark, S. D. (1996) "Cablebolting in Underground Mine". BiTech Publishers.

John, C. M. and Dillen, D. F. V. (1983) "Rock Bolts – A new numerical representation and its application in tunnel design", Proc. of 24th U. S. Symposium on Rock Mechanics, pp. 13-25.

Kılıc, A., Yasar, E. and Atis, C. D. (2003) "Effect of bar shape on the pull-out capacity of fully-grouted rockbolts" Tunnelling and Underground Space Technology, 18(1), pp. 1-6.

Kaiser, P. K. and Cai, M. (2013) "Critical review of design principles for rock support in burst-prone ground – time to rethink!", Ground Support 2013, Perth, Australia.

Kiyohara, C., Hasegawa, H. and Suenaga, S. (1999) "Studies on the mechanical properties of concrete -Mortar strength and drying shrinkage- (in Japanese)", Architectural Institute of Japan, Kyushu branch, pp. 85-88.

Li, C. C. and Stillborg, B. (1999) "Analytical models for rock bolts", International Journal of Rock Mechanics and Mining Science, 36, pp. 1013-1029.

Li, C. C. (2010) "A new energy-absorbing bolt for rock support in high stress rock masses", International Journal of Rock Mechanics and Mining Sciences, 47(3), pp. 396-404.

Li, C. C. (2011) "Performance of D-bolts Under Static Loading", Rock Mechanics and Rock Engineering, 45(2), pp. 183-192.

- Li, C. C., Stjern, G. and Arne, M. (2014) “A review on the performance of conventional and energy-absorbing rockbolts”, Journal of Rock Mechanics and Geotechnical Engineering, 6(4), pp. 315-327.
- Li, C. C. (2017) “Principles of rockbolting design”, Journal of Rock Mechanics and Geotechnical Engineering, 9(3), pp.396-414.
- Li, C. C. (2018) “Dynamic rock support in burst-prone rock masses”, EUROCK2018, Russia.
- Liu, N., Li, M., Chen, W. (2017) “Mechanical deterioration of rock salt at different confinement levels: a grain-based lattice scheme assessment”, Computers and Geotechnics, 84, pp. 210–224.
- Li, X. (2007) “Finite element modelling of skewed reinforced concrete bridges and the bond-slip relationship between concrete and reinforcement”, Master thesis, Auburn University.
- Lim, H. G., Maeda, K. and Murakami, M. (1998) “Study on Mechanical Properties of Concrete in Early Ages by Triaxial Compression Test”, Proc. of the Japan Concrete Institute, Japan Concrete Institute. 20, pp. 577-582.
- Lutz, L. A. (1970), “Analysis of stresses in concrete near a reinforcing bar due to bond and transverse cracking”, ACI Journal, pp. 778-787.
- Ma, S., Zhao, Z. Y., Nie, W. and Gui, Y. L. (2016) “A numerical model of fully grouted bolts considering the tri-linear shear bond–slip model”, Tunnelling and Underground Space Technology, 54, pp. 73-80.
- Nicolas, M., John, D. and Belytschko, T. (1999) “A finite element method for crack growth without remeshing”, International Journal for Numerical Methods in Engineering, 46, pp. 131-150.
- Nie, W., Zhao, Z. Y., Ning, Y. J. and Sun, J. P. (2014) “Development of Rock Bolt Elements in Two-Dimensional Discontinuous Deformation Analysis”, Rock Mechanics and Rock Engineering, 47(6), pp. 2157-2170.

Nie, W., Zhao, Z. Y., Ning, Y. J. and Guo W. (2014) “Numerical studies on rockbolts mechanism using 2D discontinuous deformation analysis”, Tunnelling and Underground Space Technology, 41, pp. 223-233.

Nie, W., Zhao, Z. Y., Yokota, Y. and Guo, W. (2018) “Bond-slip modelling of the rockbolt element using 2D-DDA method”, Submitted to Tunnelling and Underground Space Technology.

Ning, Y. J., Yang, J., An, X. and Ma, G. (2011) “Modelling rock fracturing and blast-induced rock mass failure via advanced discretisation within the discontinuous deformation analysis framework”, Computers and Geotechnics, 38(1), pp. 40-49.

Noguchi, H. (1981) “Nonlinear FE analysis of reinforced concrete beam-column joints”, IABSF Colloquium, pp. 567-582.

Novak, M. J. et al. (2014) “Use of alternative scanning devices for creation of 3D models of machine partes”, Technical gazette, 21, pp. 177-181.

Obara, Y., Maeno, Y., Kawano, R., Ito, F. and Nakahara, F. (2002) “Bond failure of rock bolts and cable bolts in a pull-out test”, Journal of Japan Society of Civil Engineers, III-59(708), pp. 97-106.

Ortlepp, W. D. (1992) “The design of support for the containment of rockburst damage in tunnels e an engineering approach”, Proc. of International Symposium on Rock Support in Mining and Underground Construction, pp. 593-609.

Plauk, G. and Hees, G. (1981), “FE analysis of reinforced concrete beams with special regard to bond behavior”, IABSF Colloquium, pp. 655-670.

Rabcewicz, L. v. (1964) “The New Austrian Tunnelling Method”, WATER POWER. 16, pp. 453-457.

Rabcewicz, L. v. (1964) “The New Austrian Tunnelling Method”, WATER POWER. 16, pp. 511-515.

Rabcewicz, L. v. (1965) “The New Austrian Tunnelling Method”, WATER POWER, 17, pp. 19-24.

Rehbock, S. M. and Jesel T. (2018) “Fault induced rock bursts and micro-tremors – Experiences from the Gotthard Base Tunnel”, Tunnelling and Underground Space Technology, 81, pp. 358-366.

Sabo & Landslide Technical Center (STC) (2002) “Soil -erosion control technology and Technical review and Certification committee, Permanent anchor method (SSL-CE type)”.

Schubert, W. and Moritz, B. (1998) “Controllable Ductile Support System for Tunnels in Squeezing rock”, Essen, 16, pp. 224-227.

Shafer, H. (1975) “Contribution to the solution of contact problems with the aid of bond elements”, Computation Methods in Applied Mechanics and Engineering, North Holland Publish Comapany.

Shang, J., Zhao, Z. Y., Hu, J. and Handley, K. (2018) “3D Particle-Based DEM Investigation into the Shear Behaviour of Incipient Rock Joints with Various Geometries of Rock Bridges”, Rock Mechanics and Rock Engineering, 51, pp. 3563-3584. <https://doi.org/10.1007/s00603-018-1531-0>.

Shi, G. H. (1991) “Manifold method of material analysis”, Transactions of the Ninth Army Conference on Applied Mathematics and Computing, pp. 57-76.

Shi, G. H (1988) “Block system modeling by Discontinuous Deformation Analysis”, Ph.D Thesis, University of California, Berkeley.

Steiner, W. (1996) “Tunnelling in Squeezing Rocks Case Histories”, Rock Mechanics and Rock Engineering, 29(4), pp. 211-246.

Stillborg, B. (1994) “Professional users handbook for rock bolting”, Trans Tech Publications.

Tepfers, R. (1973) “A Theory of Bond Applied to Overlapped Tensile Reinforcement Splices for Deformed Bars”, Ph.D Thesis, Chalmers University of Technology.

Tepfers, R. (1979), “Cracking of concrete cover along anchored deformed reinforcing bars”, Magazine of Concrete Research, 31, pp. 3-12.

Thielicke, W. and Stamhuis, E. J. (2014) “PIVlab – Towards User-friendly, Affordable and Accurate Digital Particle Image Velocimetry in MATLAB”, Journal of Open Research Software 2.

Tsuchiya, T. (1985) “Studies on the parameters of supports and grounds for the tunnel design program”, Journal of Japan Society of Civil Engineers, 364, pp. 31-40.

Varden, R., Lachenicht, R., Player, J., Thompson, A. and Villaescusa, E. (2008) “Development and Implementation of the Garford Dynamic Bolt at the Kanowna Belle Mine”, Tenth Underground Operators' Conference, Launceston, pp. 95-102.

Windsor, C. R. (1997) “Rock Reinforcement Systems”, International Journal of Rock Mechanics and Mining Science, 34(6).

Windsor, C. R. (1992) “Cable bolting for underground and surface excavations”, Proc. of International Symposium on Rock Support in Mining and Underground Construction.

Winkelbach, S., Molkenstruck, S. and Friedrich, M. W. (2006) “Low cost Laser Range Scanner and fast surface registration approach”, DAGM 2006, pp. 718-728.

White, D. J., Take, W. A. and Bolton M. D. (2003) “Soil deformation measurement using particle image velocimetry (PIV) and photogrammetry”, Geotechnique, 53(No.7), pp. 619-631.

Xie, Y., Cao, P., Liu, J. and Dong, L. (2016) “Influence of crack surface friction on crack initiation and propagation: A numerical investigation

based on extended finite element method”, Computers and Geotechnics, 74, pp. 1-14.

Xie, Y., Cao, P., Jin, J. and Wang, M. (2017) “Mixed mode fracture analysis of semi-circular bend (SCB) specimen: A numerical study based on extended finite element method”, Computers and Geotechnics, 82, pp. 157-172.

Yap, L. P. and Rodger, A. A. (1984) “A study of the behaviour of vertical rock anchor using the F.E.M”, International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts. Vol.21, No.2, pp. 47-61.

Yazici, S. and Kaiser, P. K. (1992) “Bond Strength of Grouted”, Int. J. Rock Mech. Min. Sci., 29(3), pp. 279-292.

Zhao, G. F. and Xia, K. (2018) “A study of mode-I self-similar dynamic crack propagation using a lattice spring model”, Computers and Geotechnics, 96, pp. 215-225.

Zhao, Z. Y., An, X. M. and Zhou, Y. X. (2013) “DDA/NMM developments and applications in Nanyang Technological University, Singapore”, 11th International Conference on Analysis of Discontinuous Deformation, Fukuoka, Japan.