1996

Application of rock mass characterisation to slope stability problems

Gholam Reza Khanlari
University of Wollongong
APPLICATION OF ROCK MASS CHARACTERISATION TO SLOPE STABILITY PROBLEMS

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

DOCTOR OF PHILOSOPHY

from

UNIVERSITY OF WOLLONGONG

By

GHOLAM REZA KHANLARI, B.Sc, M.Sc (Engineering Geology)

DEPARTMENT OF CIVIL AND MINING ENGINEERING

1996
IN THE NAME OF GOD

This thesis is dedicated to my dear parents;

My mother, Zahra Shaani
My father, Aliaqharr Khanlari

and my family

My wife, Fatemeh Kaviani Malayeri
My kids, Zahedeh, Elham and Hossein Khanlari

for their love and support
AFFIRMATION

I hereby certify that the work presented in the thesis entitled “Application of Rock Mass Characterisation to Slope Stability Problems” for fulfillment of the requirements for the award of the degree of Doctor of Philosophy, submitted in the Department of Civil and Mining Engineering, University of Wollongong, is my own work carried out under the supervision of Prof. R. N. Singh and A/ Prof. R. N. Chowdhury. The thesis contains no material previously published or written by another person except where due reference is made through the text.

GHOLAMREZA KHANLARI

The following publication has been based on this research:

ACKNOWLEDGMENTS

The author would like to express his sincere gratitude to Professor R. N. Singh, the former Head of the Department of Civil and Mining Engineering at the University of Wollongong for his supervision, encouragement and guidance and also provision of the laboratory facilities. His sheer support during laboratory work and field investigation enabled the author to carry out this research. The author would also like to express his sincere thanks to Associate Professor R. N. Chowdhury, Head of Department of Civil and Mining Engineering at the University of Wollongong, for his co-supervision, guidance, encouragement and constructive criticism.

The author also wishes to express his sincere thanks for helpful contributions made by the following professionals during the period of this research.

Dr Ian Wallace, Geology Manager of Boral Ltd for his permission to carry out the field investigation at Mugga II, Marulan and Dunmore Quarries as well as providing access to technical reports.

Mr B. Marshall, Manager Marulan Quarry (Large limestone Quarry), Mr J. Clayton, Manager Dunmore Quarry (Basalt Quarry), and Mr J. Miners, Manager Mugga II Quarry (Porphyry Quarry) for their assistance during the field investigation.

Dr J. Schonhardt, Laboratory Manager, and Mr Ian Laird of the Rock Mechanics Laboratory and all other technical staff in the Department of Civil and Mining Engineering at the University of Wollongong, for their support during this research.

Dr V. S. Vutukuri, Director Research and Postgraduate Studies, and Mr. Michael Mallos, Technical Officer of Rock Mechanics Laboratory in the School of Mines at the University of NSW for their help in executing laboratory work using the servo control testing machine.

The author also wishes to acknowledge Dr J. Hematian, Dr M. Tavakoli, and PhD student Mr A. Herath for their valuable assistance during his field investigation. Thanks to Mr Derek Hanley co-ordinator of Postgraduate Resource Center, University of Wollongong, for his comments on linguistic aspects of the thesis.

The author would like to express his special thanks to all members of his family particularly his parents and his wife, Fatemeh and his daughters, Zahedeh, Elham, and Hossna, for their support, encouragement and professional development over the years.

The author would also like to acknowledge with sincere appreciation, the financial support of the Ministry of Culture and Higher Education of the Islamic Republic of Iran and the University of Bu-Ali Sina.
The stability of hard rock slopes is a critical problem in surface mining and is governed by the presence of geological structures such as, joints, fractures, faults, shear zones and bedding planes along which failure may occur. The accurate prediction of the behaviour of intact rock and rock masses is of the utmost importance for stability analysis of surface mining excavations and also for safer mining operations. The engineering judgment of the relationship between geological parameters and physical and mechanical properties of intact rock and also rock mass discontinuities in conjunction with the slope stability analysis, requires knowledge of engineering geology, comprehensive laboratory testing and field investigations of the rock mass and surrounding formations.

The main purpose of rock slope stability analysis is to obtain a safer and more economic design for mining operations. Surface mining is one of the most common methods used for the extraction of stones from quarries and other near surface mineral deposits which are not able to be economically extracted by underground techniques. Therefore, surface mining and particularly hard rock slope instability problems are very important from point of view of mining operations.

The main objective of this research work is to study the relationship between rock slope stability problems and the mechanical properties of jointed rock mass in surface mines. In this research, theoretical, empirical and analytical techniques related to slope stability analysis were reviewed and it was found that the three-dimensional scanline method is more suitable for joint survey and data collection from the rock mass discontinuities.

This research describes discontinuity data analysis and presentation methods, which makes it possible to define the major joint sets from the discontinuities survey for the purpose of slope stability analysis in surface mining.

The objectives of this thesis can be summarised as follows:

- To study the significant factors affecting rock slope stability, particularly the effects of discontinuities and their orientation on modes of failure.
• To perform a statistical evaluation of the physical and mechanical properties of discontinuities and consideration of more significant factors affecting rock slope stability.

• To evaluate the physical and mechanical properties of intact rock using laboratory data in order to estimate rock mass characteristics.

• To define the interaction between the physical and mechanical properties of intact rock with that of discontinuities and their effects on the rock mass slope stability.

• Rock mass classification and determination of rock mass quality.

• Case studies of the three different types of hard rocks from different surface mines in Australia.

Field investigations have been carried out in three hard rock quarries to obtain information regarding the geology of the sites, and the characteristics of the main type of rocks. This investigations have been carried out on three different types of open pit mines namely Mugga II, Marulan, and Dunmore quarries in New South Wales, Australia for the assessment of stability of slope faces in these quarries. The mineralogical composition, as well as structural geology of the Porphyry rock, Basalt, and Limestone were investigated. Laboratory tests have been carried out on different types of rocks from the quarries to acquire data on the engineering properties of intact rock and to characterise the rock masses. The factors affecting the shear strength of discontinuity surfaces and an interpretation of the shear strength of natural joints together with discontinuity data collected from the rock masses are presented in chapter 4. A description of the relevant features of the quarries are included, and the results are presented in chapters 5, 6, and 7.

The research for this thesis was carried out in three major steps. A site investigation was carried out on three different hard rock quarries to establish their geological and geotechnical natures, and to assess significant factors affecting rock slope stability. In addition, data from the physical and mechanical properties of discontinuities were used in the preparation of input data for the numerical analysis of discontinuities. During the testing program more than 300 samples were prepared from the block samples carried from the three sites. They were subjected to uniaxial and triaxial compressive strength, point load test and Brazilian tests in order to correlate these properties with the shear strength of the masses. In conjunction with the direct shear test some samples, containing natural joints, were also collected from sites for the purpose of testing and data analysis.
The aim of testing program was to obtained rock mass classification data and also to establish correlation between shear strength parameters (C and \( \phi \)) with the testing results.

Data from geological mappings (joint survey) were examined using the stereographic projection method for the stability analysis of slope faces from the relevant parts of the quarries. At this stage a computer program was used for the plotting of the poles, contoured pole nets, the presentation of the joint systems (main joint sets), friction cone and slope faces on a projection net (a lower hemispherical projection) to establish the instability potential of the quarries slope faces.

Rock mass classification has been carried out on three different types of rock masses in order to determine the in-situ properties of rock masses based on the classification index and laboratory results. From the application of different rock mass classification system it was concluded that the rating given to the uniaxial compressive strength and spacing are not adequate. Therefore, a new rating scheme was developed and proposed for the RMR classification system.

Based on field investigations it has been identified that the three-dimensional joint survey is a suitable and cost effective method for collecting data from the discontinuities within the rock mass. The results of direct shear test showed that both linear and power law criteria are close and parallel to each other and they show a very high correlation coefficient with the experimental results.

Some of the regression methods were examined on various laboratory and index tests and the results showed that there is a very good relationship between the laboratory and index tests for different types of hard rock. A statistical analysis of discontinuity parameters showed that the instability and modes of failure in hard rock slopes are mainly controlled by discontinuity length.
CHAPTER ONE: ROCK SLOPE STABILITY PROBLEMS IN HARD ROCK QUARRIES

1.1 INTRODUCTION ................................................................. 1

1.2 CLASSIFICATION OF SURFACE MINING METHODS ...................... 2
  1.2.1 Open Pit Mining ....................................................... 3
  1.2.2 Quarrying ............................................................ 3
  1.2.3 Open Cast Mining .................................................... 3
  1.2.4 Auger Mining Method ............................................. 3

1.3 QUARRYING AND SURFACE MINING OPERATIONS IN AUSTRALIA .... 4

1.4 REVIEW OF THE SITES IN NEW SOUTH WALES ......................... 6

1.5 TYPICAL MODE OF FAILURES IN SOFT AND HARD ROCKS ............ 8
  1.5.1 Soil and Soft Rock Slopes ....................................... 9
  1.5.2 Hard Rock Slopes .................................................. 10

1.6 FACTORS AFFECTING THE STABILITY OF ROCK SLOPES ................. 11
  1.6.1 The Influence of Internal Factors on Rock Slope Stability ........ 12
  1.6.2 The Influence of the External Factors on Rock Slope Stability .... 13
  1.6.3 The Influence of Vibration on the Stability of Hard Rock Slopes .... 14
  1.6.4 The Influence of Drilling and Blasting on the Slope Stability of Hard Rocks . 15
  1.6.5 The Human Safety Problems Due to Open Pit Mining ............... 15
  1.6.6 Mine Operation and Economic Problems ........................... 16
CHAPTER TWO: EFFECTS OF ROCK MASS CHARACTERISTICS ON SLOPE STABILITY

2.1 INTRODUCTION ........................................................................................................... 24

2.2 FACTORS AFFECTING THE ASSESSMENT OF ROCK MASSES FOR ENGINEERING PURPOSES ........................................................................................................ 25
  2.2.1 Geological Factors ................................................................................................. 26
  2.2.2 Distribution of Regional Stresses ........................................................................... 26
  2.2.3 Hydrological Conditions ....................................................................................... 26

2.3 PHYSICAL CHARACTERISTICS OF DISCONTINUITIES USED IN ROCK MASS CLASSIFICATIONS ........................................................................................................ 28
  2.3.1 Discontinuities ....................................................................................................... 28
    2.3.1.1 Different Types of Discontinuities .................................................................. 28
    2.3.1.2 Physical Characteristics of Discontinuities ..................................................... 30

2.4 SCALE EFFECT OF THE ROCK STRENGTH AND DEFORMABILITY .... 45

2.5 DIFFERENT METHODS FOR DATA COLLECTION OF DISCONTINUITIES ............................................................................................................................ 47
  2.5.1 Scan-line Method ................................................................................................. 47
  2.5.2 Area Mapping Method ......................................................................................... 47

2.6 MECHANICAL PROPERTIES OF INTACT ROCK ..................................................... 48
  2.6.1 Rock Types and Lithology ..................................................................................... 48
  2.6.2 Strength .............................................................................................................. 49
  2.6.3 Anisotropy .......................................................................................................... 50
  2.6.4 Weathering .......................................................................................................... 50

2.7 GRAPHICAL PRESENTATION OF DISCONTINUITY DATA ANALYSIS ............................................................................................................................ 51
  2.7.1 Introduction ......................................................................................................... 51
3.4.2 Extrinsic Factors Influencing Rock Strength .................................................. 91
3.4.2.1 Specimen Volume ................................................................. 91
3.4.2.2 Strain Rate ........................................................................ 91
3.4.2.3 Testing Machine ................................................................. 92
3.4.2.4 End Effects and the Influence of Length to Diameter Ratio ............... 92

3.5 ESTIMATION OF UNIAXIAL COMpressive STRENGTH OF ROCKS.. 93
3.5.1 Sample Preparation for Uniaxial Compressive Strength Test ................. 94
3.5.2 Testing Procedure .................................................................. 96
3.5.3 Discussion on the Results of Uniaxial Compression Test ........................ 102

3.6 DETERMINATION OF DEFORMATION PROPERTIES OF ROCK ....... 103
3.6.1 Young's Modulus .................................................................. 103
3.6.2 Poisson's Ratio .................................................................... 104

3.7. DETERMINATION OF THE SHEAR STRENGTH OF ROCK BY DIRECT SHEAR TEST .......................................................... 106
3.7.1 Testing Procedures ................................................................ 106
3.7.2 Determination of Friction Angle and Cohesion Factor ....................... 106

3.7.8 DETERMINATION OF STRENGTH OF ROCK BY THE TRIAXIAL TESTING METHOD ...................................................... 107
3.8.1 Preparation of Samples and Testing Procedure ............................... 108
3.8.2 Testing procedure ................................................................ 108
3.8.3 Analysis of Triaxial Compression Test Result .................................. 108

3.9 BRAZILIAN TEST FOR DETERMINATION OF THE TENSILE STRENGTH OF ROCK ................................................................. 116
3.9.1 Testing Methods .................................................................... 116
3.9.2 Results of Brazilian Tests and Discussion ...................................... 117

3.10 POINT LOAD TEST, INDEX STRENGTH TEST ........................................... 119
3.10.1 Testing Methods ................................................................... 124
3.10.2 Results of Point Load Test and Discussion .................................... 127

3.11 DETERMINATION OF STRENGTH OF ROCK BY SCHMIDT HAMMER .......................................................... 127

3.12. LABORATORY TESTS CORRELATIONS FOR HARD ROCKS ............... 136
3.12.1 Uniaxial Compressive Strength and Diametral Point Load Index Test .... 136
3.12.2 Uniaxial Compressive Strength and Axial Point Load Test: .................. 138
5.2 SITE INVESTIGATIONS AND GEOLOGICAL STUDY ........................................... 201
  5.2.1 Geomorphology of Canberra ................................................................. 201
  5.2.2 Mugga Mugga Porphyry Member ........................................................... 202

5.3 Geological Structures .................................................................................. 205

5.4 STATISTICAL ANALYSIS OF DISCONTINUITY DATA FROM
MUGGA II QUARRY .......................................................................................... 205
  5.4.1 Descriptive statistical analysis ................................................................. 207
    5.4.1.1 Orientation of Discontinuities ......................................................... 208
    5.4.1.2 Dip Angle ......................................................................................... 209
    5.4.1.3 Discontinuity Aperture .................................................................... 212
    5.4.1.4 Discontinuity Infilling Materials .................................................... 213
    5.4.1.5 Joint Compressive Strength .............................................................. 216
    5.4.1.6 Discontinuity Water Condition ......................................................... 216
    5.4.1.7 Discontinuity Curvature .................................................................. 218
    5.4.1.8 Discontinuity Roughness ................................................................ 220
    5.4.1.9 Discontinuity Persistence ................................................................. 222
    5.4.1.10 Discontinuity Spacing ................................................................. 227
    5.4.1.11 Rock Quality Designation (RQD) .................................................. 232
    5.4.1.12 Discontinuity Trace Length .......................................................... 237

5.5 ANALYTICAL STATISTICS .......................................................................... 241
  5.5.1 Data Acquisition and Factor Analysis Propose ....................................... 241

5.6 DIFFERENT MODES OF FAILURES ......................................................... 243
  5.6.1 Plane Mode of Failure ........................................................................... 243
  5.6.2 Wedge Failure ......................................................................................... 245
  5.6.3 Circular Failure ....................................................................................... 246
  5.6.4 Toppling Failure ..................................................................................... 246

5.7 THE EFFECT OF DISCONTINUITIES IN SLOPE STABILITY ..................... 248

5.8 DISCONTINUITY DATA COLLECTION AND ANALYSIS ......................... 248

5.9 GRAPHICAL PRESENTATION OF DISCONTINUITIES .............................. 249
  5.9.1 Stability Assessment of the Slope Face in Section A .............................. 249
  5.9.2 Stability Assessment of the Slope Face in Section B .............................. 250
  5.8.3 Stability Assessment of the Slope Face in Section C .............................. 255
  5.8.4 Stability Assessment of The Slope Face in Section D ............................ 255

5.10 ENGINEERING PROPERTIES OF INTACT ROCK ................................. 261
5.11 SCALE EFFECT ON THE STRENGTH AND ELASTIC MODULUS OF ROCKS .................................................................................. 261
5.12 EVALUATION OF ROCK MASS STRENGTH PARAMETERS .................................................................................................. 262
  5.12.1 Application of Rock Mass Quality System (RMR) ................................................................. 262
  5.12.2 Application of Q System............................................................................................................. 265
  5.12.3 Application of Weakening Coefficient Classification System (WC) ............................... 265
5.13 ENGINEERING DESCRIPTION OF THE PORPHYRY ROCK MASS IN MUGGA II QUARRY .............................................................................. 268
5.14 CONCLUSIONS ......................................................................................................................................................... 269

CHAPTER SIX: GEOLOGICAL INVESTIGATION AND APPRAISAL OF SLOPE STABILITY OF A LIMESTONE QUARRY
6.1 INTRODUCTION ................................................................................................................................................. 270
6.2 GENERAL GEOLOGY AND SITE INVESTIGATIONS ....................................................................................... 271
6.3 GEOLOGY OF THE MARULAN QUARRY ................................................................................................................. 272
6.4 STRATIGRAPHY OF BUNGONIA LIMESTONE .............................................................................................. 275
6.5 MAIN LIMESTONE BODY .................................................................................................................................... 277
6.6 GEOHYDROLOGICAL CONDITION ......................................................................................................................... 279
6.7 RESULTS OF FIELD OBSERVATIONS AND GEOLOGICAL STUDIES .............................................................. 280
6.8 METHODOLOGY OF DISCONTINUITY DATA COLLECTION AND ANALYSIS ................................................................. 280
6.9 STATISTICAL ANALYSIS OF DISCONTINUITY DATA FROM MARULAN QUARRY .......................................................... 281
  6.9.1 Introduction .................................................................................................................................................. 281
  6.9.2 Descriptive Statistical Analysis .................................................................................................................... 281
    6.9.2.1 Discontinuity Orientation .................................................................................................................... 281
    6.9.2.2 Dip Angle of Discontinuities .............................................................................................................. 282
    6.9.2.3 Discontinuity Aperture ....................................................................................................................... 283
    6.9.2.4 Discontinuity Infilling Materials ........................................................................................................ 283
    6.9.2.5 Discontinuity Water Condition .......................................................................................................... 284
    6.9.2.6 Discontinuity Curvature ..................................................................................................................... 287
    6.9.2.7 Discontinuity Roughness ................................................................................................................... 287
    6.9.2.8 Discontinuity Persistence ................................................................................................................... 289
6.9.2.9 Discontinuity Spacing .................................................. 289
6.9.2.10 Joint Compressive Strength (JCS) ................................. 290
6.9.2.11 Rock Quality Designation (RQD) ................................. 293
6.9.2.12 Discontinuity Length .................................................. 294

6.10 ANALYTICAL STATISTICS ............................................................................. 296
6.10.1 Factor Analysis and Data Acquisition .............................................. 296

6.11 APPRAISAL OF FACE STABILITY AT A LIMESTONE QUARRY IN MARULAN ................................................................. 296
6.11.1 Stability Assessment of the Slope Face in Section A ....................... 297
6.11.2 Stability Assessment of the Slope Face in Section B ....................... 300

6.12 EVALUATION OF ROCK MASS STRENGTH PROPERTIES ......................................................................................... 303

6.13 ROCK MASS CHARACTERISATION AT DUNMORE QUARRY ......................................................................................... 304
6.13.1 Engineering Properties of Intact Rock ........................................... 304
6.13.2 Evaluation of the Rock Mass Strength Parameters ....................... 305
6.13.3 Application of Rock Mass Classification Systems ....................... 305

6.14 SUMMARY OF ENGINEERING DESCRIPTION OF THE LIMESTONE ROCK MASS IN MARULAN QUARRY ................................................. 309

6.15 CONCLUSIONS ............................................................................................. 310

CHAPTER SEVEN: APPLICATION OF MODIFIED ROCK MASS CLASSIFICATION SYSTEMS ON STABILITY OF BASALT QUARRY

7.1 INTRODUCTION ............................................................................................. 311
7.2 DUNMORE QUARRY: GENERAL INFORMATION ....................................... 311
7.3 PREVIOUS WORKS ...................................................................................... 315
7.4 GENERAL GEOLOGY IN DUNMORE QUARRY ........................................... 317
7.5 GEOLOGY AND SITE INVESTIGATION IN DUNMORE QUARRY .............. 317
7.6 STRUCTURAL GEOLOGY ............................................................................. 320
7.7 STATISTICAL ANALYSIS OF DISCONTINUITY DATA IN DUNMORE QUARRY ............................................................................. 320
7.7.1 INTRODUCTION ...................................................................................... 320
7.8 METHODOLOGY OF DISCONTINUITY DATA COLLECTION AND ANALYSIS ..................................................................................... 322
7.9 DESCRIPTIVE STATISTICS

7.9.1 Discontinuity Dip Angle ........................................... 322
7.9.2 Discontinuity Aperture ............................................ 323
7.9.3 Orientation of Discontinuities .................................. 325
7.9.4 Water Condition .................................................. 326
7.9.5 Discontinuity Curvature .......................................... 326
7.9.6 Discontinuity Roughness ........................................ 328
7.9.7 Discontinuity Persistence ........................................ 329
7.9.8 Discontinuity Length ............................................. 332
7.9.10 Discontinuity Spacing .......................................... 334
7.9.11 Rock Quality Designation (RQD) ............................ 339

7.10 ANALYTICAL STATISTICS .......................................... 341

7.10.1 Factor Analysis and Data Acquisition ........................ 341

7.11 STABILITY ASSESSMENT AND GRAPHICAL PRESENTATION OF DISCONTINUITIES ......................................................... 343

7.11.1 Stability Assessment of the Slope Face in Section A ........ 343
7.11.2 Stability Assessment of the Slope Face in Section B ........ 347

7.12 ROCK MASS CHARACTERISATION AT DUNMORE QUARRY .... 350

7.12.1 Engineering Properties Of Intact Rock ....................... 351
7.12.2 Estimation of Rock Mass Strength Parameters .............. 352
7.12.3 Rock Mass Classification of Dunmore Quarry .............. 352
7.12.3.1 Results of Geomechanics Classification System (RMR) ...... 352
7.12.3.2 Results of Rock Mass Quality (Q System) .................. 353
7.12.3.3 Results of Weakening Coefficient Classification System ..... 355

7.13 SUMMARY OF ENGINEERING EVALUATION OF THE ROCK MASS IN DUNMORE QUARRY ............................................... 357

7.14 CONCLUSIONS .......................................................... 358

CHAPTER EIGHT: CONCLUSIONS AND RECOMMENDATIONS

8.1 SUMMARY .............................................................. 359
8.2 CONCLUSIONS .......................................................... 360

ix
8.3 APPLICATION AND MODIFICATION OF ROCK MASS CLASSIFICATION SYSTEMS ................................................................. 361
8.4 FIELD INVESTIGATION .............................................................................................................................................. 363
8.5 LABORATORY TESTING OF DIFFERENT TYPE OF ROCKS ......................................................................................... 363
8.6 STATISTICAL ANALYSIS OF LABORATORY AND FIELD DATA .............................................................................. 364
8.7 STABILITY ASSESSMENT OF ROCK SLOPE FACES .............................................................................................. 365
8.8 RECOMMENDATIONS FOR FUTURE RESEARCH ............................................................................................... 366

REFERENCES ................................................................................................................................................................. 368
BIBLIOGRAPHY ............................................................................................................................................................ 389
APPENDIX 1 ................................................................................................................................................................. 402
APPENDIX 2 ................................................................................................................................................................. 406
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure No.</th>
<th>Description</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>Location of study sites in NSW, Australia</td>
<td>12</td>
</tr>
<tr>
<td>1.2</td>
<td>Slope stability flowchart for the purpose of surface mining</td>
<td>18</td>
</tr>
<tr>
<td>1.3</td>
<td>Pattern of thesis structure</td>
<td>22</td>
</tr>
<tr>
<td>2.1</td>
<td>Different forces on a detached block on rock slope</td>
<td>27</td>
</tr>
<tr>
<td>2.2</td>
<td>Typical joint survey data sheet for evaluation of discontinuity characteristics</td>
<td>29</td>
</tr>
<tr>
<td>2.3</td>
<td>Illustration of number of joint sets</td>
<td>31</td>
</tr>
<tr>
<td>2.4</td>
<td>Illustration of dip direction ( \alpha ) and dip magnitude ( \beta )</td>
<td>32</td>
</tr>
<tr>
<td>2.5</td>
<td>Diagram indicating the strike, dip and dip direction of three different oriented planes</td>
<td>33</td>
</tr>
<tr>
<td>2.6</td>
<td>Illustration of typical roughness profiles</td>
<td>37</td>
</tr>
<tr>
<td>2.7</td>
<td>Joint spacing measurement of rock mass</td>
<td>38</td>
</tr>
<tr>
<td>2.8</td>
<td>Persistence of the various sets of discontinuities</td>
<td>40</td>
</tr>
<tr>
<td>2.9</td>
<td>Sketches of rock masses illustrating (a) Blocky, (b) Irregular, (c) Tabular, and (d) Columnar block shapes</td>
<td>42</td>
</tr>
<tr>
<td>2.10</td>
<td>Illustration of the effect of major and minor joint system on the stability of rock slopeaces in an open pit mine</td>
<td>46</td>
</tr>
<tr>
<td>2.11</td>
<td>Definition of terms used in conjunction with the lower reference hemisphere stereographic projection</td>
<td>54</td>
</tr>
<tr>
<td>2.12</td>
<td>Method of construction of an equal area (a) and equal angle projection (b)</td>
<td>55</td>
</tr>
<tr>
<td>2.13</td>
<td>Stereographic projection net</td>
<td>56</td>
</tr>
<tr>
<td>2.14</td>
<td>A polar stereographic net</td>
<td>56</td>
</tr>
<tr>
<td>2.15</td>
<td>An equal area net</td>
<td>57</td>
</tr>
<tr>
<td>2.16</td>
<td>A Denness counting net</td>
<td>57</td>
</tr>
<tr>
<td>2.17</td>
<td>Effect of ground water table on soil and rock slope stability</td>
<td>59</td>
</tr>
<tr>
<td>3.1</td>
<td>Illustration of flexible mould used in casting of the block samples</td>
<td>85</td>
</tr>
<tr>
<td>3.2</td>
<td>Illustration of samples cast in concrete</td>
<td>85</td>
</tr>
<tr>
<td>3.3</td>
<td>Coring machine used for preparation of cylindrical specimens</td>
<td>88</td>
</tr>
<tr>
<td>3.4</td>
<td>Illustration of grinding machine used for grinding the ends of specimens</td>
<td>88</td>
</tr>
<tr>
<td>Figure No.</td>
<td>Page No.</td>
<td></td>
</tr>
<tr>
<td>-----------</td>
<td>----------</td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td>97</td>
<td></td>
</tr>
<tr>
<td>3.6</td>
<td>97</td>
<td></td>
</tr>
<tr>
<td>3.7</td>
<td>98</td>
<td></td>
</tr>
<tr>
<td>3.8</td>
<td>99</td>
<td></td>
</tr>
<tr>
<td>3.9</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>3.10</td>
<td>101</td>
<td></td>
</tr>
<tr>
<td>3.11</td>
<td>110</td>
<td></td>
</tr>
<tr>
<td>3.12</td>
<td>111</td>
<td></td>
</tr>
<tr>
<td>3.13</td>
<td>112</td>
<td></td>
</tr>
<tr>
<td>3.14</td>
<td>113</td>
<td></td>
</tr>
<tr>
<td>3.15</td>
<td>114</td>
<td></td>
</tr>
<tr>
<td>3.16</td>
<td>115</td>
<td></td>
</tr>
<tr>
<td>3.17</td>
<td>118</td>
<td></td>
</tr>
<tr>
<td>3.18</td>
<td>118</td>
<td></td>
</tr>
<tr>
<td>3.19</td>
<td>125</td>
<td></td>
</tr>
<tr>
<td>3.20</td>
<td>125</td>
<td></td>
</tr>
<tr>
<td>3.21</td>
<td>137</td>
<td></td>
</tr>
<tr>
<td>3.22</td>
<td>138</td>
<td></td>
</tr>
<tr>
<td>3.23</td>
<td>139</td>
<td></td>
</tr>
<tr>
<td>3.24</td>
<td>140</td>
<td></td>
</tr>
<tr>
<td>3.25</td>
<td>141</td>
<td></td>
</tr>
<tr>
<td>3.26</td>
<td>142</td>
<td></td>
</tr>
<tr>
<td>3.27</td>
<td>143</td>
<td></td>
</tr>
<tr>
<td>3.28</td>
<td>144</td>
<td></td>
</tr>
<tr>
<td>3.29</td>
<td>144</td>
<td></td>
</tr>
<tr>
<td>Figure No.</td>
<td>Page No.</td>
<td></td>
</tr>
<tr>
<td>-----------</td>
<td>----------</td>
<td></td>
</tr>
<tr>
<td>3.30</td>
<td>Linear regression between diametral point load index and axial point load index tests in hard rocks</td>
<td>1145</td>
</tr>
<tr>
<td>3.31</td>
<td>Power regression between diametral point load index and axial point load index tests in hard rocks</td>
<td>146</td>
</tr>
<tr>
<td>4.1</td>
<td>Preparation of specimen for direct shear test</td>
<td>152</td>
</tr>
<tr>
<td>4.2</td>
<td>Bi-linear failure envelope for multiple inclined surfaces</td>
<td>160</td>
</tr>
<tr>
<td>4.3</td>
<td>Preparation of shear strength curves for three types of failure in heavily jointed, interlocking rock mass</td>
<td>164</td>
</tr>
<tr>
<td>4.4</td>
<td>Representation of basic assumptions and expected results for the proposed failure model</td>
<td>169</td>
</tr>
<tr>
<td>4.5</td>
<td>Typical profiles of roughness in associated JRC Values</td>
<td>173</td>
</tr>
<tr>
<td>4.6</td>
<td>Representation of Barton’s prediction for the roughness shear strength of discontinuities</td>
<td>174</td>
</tr>
<tr>
<td>4.7</td>
<td>Illustration of Contour gauge used for measuring the roughness of joint surfaces</td>
<td>174</td>
</tr>
<tr>
<td>4.8</td>
<td>Relationship between shear strength-versus normal strength for typical rock surface</td>
<td>175</td>
</tr>
<tr>
<td>4.9</td>
<td>Relationship between shear stress-versus effective normal stress</td>
<td>176</td>
</tr>
<tr>
<td>4.10</td>
<td>Peak shear strength envelope for natural joint samples in sections A and B of Mugga II Quarry</td>
<td>177</td>
</tr>
<tr>
<td>4.11</td>
<td>Peak shear strength envelope for natural joint samples in sections C and D of Mugga II Quarry</td>
<td>178</td>
</tr>
<tr>
<td>4.12</td>
<td>Peak shear strength envelope for natural joint samples of Marulan Quarry</td>
<td>179</td>
</tr>
<tr>
<td>4.13</td>
<td>Peak shear strength envelope for natural joint samples of Dunmore Quarry</td>
<td>180</td>
</tr>
<tr>
<td>4.14</td>
<td>Representation of different modes of direct shear test</td>
<td>181</td>
</tr>
<tr>
<td>4.15</td>
<td>Illustration of dilatancy derived from the sample sheared by normal stress applied</td>
<td>182</td>
</tr>
<tr>
<td>4.16</td>
<td>Classification of roughness and prediction of shear strength for non-planer rock joints</td>
<td>185</td>
</tr>
<tr>
<td>4.17</td>
<td>Cumulative means shear stress- shear displacement (a) and vertical displacement versus shear displacement “dilatation”(b)</td>
<td>187</td>
</tr>
<tr>
<td>Figure No.</td>
<td>Description</td>
<td>Page No.</td>
</tr>
<tr>
<td>-----------</td>
<td>-------------------------------------------------------------------------------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>4.18</td>
<td>Shear stress-shear displacement diagram for natural joints of porphyry rock from sections A and B of Mugga II Quarry</td>
<td>191</td>
</tr>
<tr>
<td>4.19</td>
<td>Peak shear strength envelope for natural joints of porphyry rock from sections A and B of Mugga II Quarry</td>
<td>192</td>
</tr>
<tr>
<td>4.20</td>
<td>Shear stress-shear displacement diagram for natural joints of porphyry rock from sections C and D of Mugga II Quarry</td>
<td>193</td>
</tr>
<tr>
<td>4.21</td>
<td>Peak shear strength envelope for natural joints of porphyry rock from sections C and D of Mugga II Quarry</td>
<td>194</td>
</tr>
<tr>
<td>4.22</td>
<td>Shear stress-shear displacement diagram for natural joints of porphyry rock from Marulan Quarry</td>
<td>195</td>
</tr>
<tr>
<td>4.23</td>
<td>Peak shear strength envelope for natural joints of porphyry rock from Marulan Quarry</td>
<td>196</td>
</tr>
<tr>
<td>4.24</td>
<td>Shear stress-shear displacement diagram for natural joints of porphyry rock from Dunmore Quarry</td>
<td>197</td>
</tr>
<tr>
<td>4.25</td>
<td>Peak shear strength envelope for natural joints of porphyry rock from Dunmore Quarry</td>
<td>198</td>
</tr>
</tbody>
</table>

| 5.1       | Locality map of the study area (Mugga II Quarry)                                                | 202      |
| 5.2       | Geomorphology map of the Canberra region                                                        | 203      |
| 5.3       | Stratigraphy of the Canberra formation                                                          | 204      |
| 5.4       | General view of Mugga II Quarry                                                                 | 206      |
| 5.5       | Rose diagram of discontinuity orientations in section A of the quarry                           | 210      |
| 5.6       | Rose diagram of discontinuity orientations in section B of the quarry                           | 210      |
| 5.7       | Rose diagram of discontinuity orientations in section C of the quarry                           | 210      |
| 5.8       | Rose diagram of discontinuity orientations in section D of the quarry                           | 210      |
| 5.9       | Probability density histogram of the discontinuity dip angle in section A                       | 211      |
| 5.10      | Probability density histogram of the discontinuity dip angle in section B                       | 211      |
| 5.11      | Frequency distribution of discontinuity dip angle in section B                                  | 211      |
| 5.12      | Probability density histogram of the discontinuity dip angle in section D                       | 211      |
| 5.13      | Frequency distribution of discontinuity aperture in section A of the quarry                     | 211      |
| 5.14      | Probability density histogram of the discontinuity aperture in section A                        | 214      |
| 5.15      | Frequency distribution of discontinuity aperture in section B                                   | 214      |
| 5.16      | Probability density histogram of the discontinuity aperture in section C                        | 214      |
5.17 Probability density histogram of the discontinuity infilling materials in section A ................................................................. 215
5.18 Probability density histogram of the discontinuity infilling material in section B ........................................................................ 215
5.19 Relative frequency of discontinuity infilling materials in section C ......................................................................................... 215
5.20 Frequency distribution of discontinuity infilling materials in section D .................................................................................... 215
5.21 Frequency distribution of JCS in section A of the quarry ............................................................................................................. 217
5.22 Probability density histogram of JCS in section B ......................................................................................................................... 217
5.23 Frequency distribution of JCS in section C of the quarry .............................................................................................................. 217
5.24 Probability density histogram of JCS in section D ......................................................................................................................... 217
5.25 Probability density histogram of discontinuity curvature in section A ........................................................................................ 219
5.26 Frequency distribution of discontinuity curvature in section B .................................................................................................. 219
5.27 Probability density histogram of discontinuity curvature in section C ........................................................................................ 219
5.28 Probability density histogram of discontinuity curvature in section D ........................................................................................ 219
5.29 Probability density histogram of discontinuity roughness in section A ....................................................................................... 221
5.30 Probability density histogram of discontinuity roughness in section B ....................................................................................... 221
5.31 Probability density histogram of discontinuity roughness in section C ....................................................................................... 221
5.32 Probability density histogram of discontinuity roughness in section D ....................................................................................... 221
5.33 Plan of discontinuity persistence into the rock mass ...................................................................................................................... 223
5.34 Illustration of joint persistence as length ratio .......................................................................................................................... 223
5.35 Frequency distribution of discontinuity termination in section A .............................................................................................. 226
5.36 Relative frequency of discontinuity termination in section B ...................................................................................................... 226
5.37 Probability density histogram of discontinuity termination in section C ..................................................................................... 226
5.38 Frequency distribution of discontinuity termination in section D .............................................................................................. 226
5.39 Frequency distribution of discontinuity spacings in section A ..................................................................................................... 230
5.40 Probability density histogram of discontinuity spacings in section B ........................................................................................ 230
5.41 Relative frequency of discontinuity spacings in section C .......................................................................................................... 230
5.42 Frequency distribution of discontinuity spacings in section D ..................................................................................................... 230
5.43 Probability density histogram of discontinuity spacings in Mugga II Quarry ........................................................................ 231
5.44 Quality description chart ......................................................................................................................................................... 236
5.45 Frequency distribution of discontinuity lengths in section A ..................................................................................................... 238
5.46 Probability density histogram of discontinuity lengths in section B ........................................................................................ 238
5.47 Probability density histogram of discontinuity lengths in section C ........................................................................................ 238
<table>
<thead>
<tr>
<th>Figure No.</th>
<th>Description</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.48</td>
<td>Frequency distribution of discontinuity lengths in section D</td>
<td>238</td>
</tr>
<tr>
<td>5.49</td>
<td>Illustration of slope geometry</td>
<td>243</td>
</tr>
<tr>
<td>5.50</td>
<td>Forces affecting stability of dry slope face of a rock mass</td>
<td>244</td>
</tr>
<tr>
<td>5.51</td>
<td>Analysing of forces affecting wedge failure</td>
<td>245</td>
</tr>
<tr>
<td>5.52</td>
<td>Forces affecting on the toppling failure</td>
<td>246</td>
</tr>
<tr>
<td>5.53</td>
<td>Discontinuity orientation data analysis of section A from Mugga II Quarry</td>
<td>251</td>
</tr>
<tr>
<td>5.54</td>
<td>Analysis of cluster poles of section A from Mugga II Quarry</td>
<td>252</td>
</tr>
<tr>
<td>5.55</td>
<td>Discontinuity orientation data analysis of section B from Mugga II Quarry</td>
<td>253</td>
</tr>
<tr>
<td>5.56</td>
<td>Analysis of cluster poles of section B from Mugga II Quarry</td>
<td>254</td>
</tr>
<tr>
<td>5.57</td>
<td>Illustration of discontinuity orientation data of section C</td>
<td>257</td>
</tr>
<tr>
<td>5.58</td>
<td>Analysis of cluster poles of section C from Mugga II Quarry</td>
<td>258</td>
</tr>
<tr>
<td>5.59</td>
<td>Analysis of discontinuity orientation data of section D</td>
<td>259</td>
</tr>
<tr>
<td>5.60</td>
<td>Analysis of cluster poles of section D from Mugga II Quarry</td>
<td>260</td>
</tr>
<tr>
<td>6.1</td>
<td>Illustration of study area and regional setting of the Bungonia area</td>
<td>271</td>
</tr>
<tr>
<td>6.2</td>
<td>Geological map of the Marulan - Windellama area</td>
<td>273</td>
</tr>
<tr>
<td>6.3</td>
<td>Geological map of the Marulan South-Bungonia</td>
<td>274</td>
</tr>
<tr>
<td>6.4</td>
<td>Stratigraphic column from the Bungonia area</td>
<td>276</td>
</tr>
<tr>
<td>6.5</td>
<td>General view of the large limestone quarry (Marulan Quarry)</td>
<td>278</td>
</tr>
<tr>
<td>6.6</td>
<td>Illustration of some dykes in the northern part of the Marulan Quarry</td>
<td>280</td>
</tr>
<tr>
<td>6.7</td>
<td>Rose diagram of discontinuity orientations in section A</td>
<td>285</td>
</tr>
<tr>
<td>6.8</td>
<td>Rose diagram of discontinuity orientations in section B</td>
<td>285</td>
</tr>
<tr>
<td>6.9</td>
<td>Probability density histogram of plane inclination in section A</td>
<td>285</td>
</tr>
<tr>
<td>6.10</td>
<td>Frequency distribution of discontinuity dip angle in section B</td>
<td>285</td>
</tr>
<tr>
<td>6.11</td>
<td>Probability density histogram of aperture in section A</td>
<td>286</td>
</tr>
<tr>
<td>6.12</td>
<td>Frequency distribution of discontinuity aperture in section B</td>
<td>286</td>
</tr>
<tr>
<td>6.13</td>
<td>Probability density histogram of discontinuity infilling material in section A</td>
<td>286</td>
</tr>
<tr>
<td>6.14</td>
<td>Probability density histogram of discontinuity infilling material in section B</td>
<td>286</td>
</tr>
<tr>
<td>6.15</td>
<td>Frequency distribution of discontinuity curvature in section A</td>
<td>288</td>
</tr>
<tr>
<td>6.16</td>
<td>Probability density histogram of discontinuity curvature in section B</td>
<td>288</td>
</tr>
<tr>
<td>6.17</td>
<td>Probability density histogram of discontinuity roughness in section A</td>
<td>288</td>
</tr>
<tr>
<td>Figure No.</td>
<td>Page No.</td>
<td></td>
</tr>
<tr>
<td>------------</td>
<td>----------</td>
<td></td>
</tr>
<tr>
<td>6.18</td>
<td>288</td>
<td></td>
</tr>
<tr>
<td>6.19</td>
<td>291</td>
<td></td>
</tr>
<tr>
<td>6.20</td>
<td>291</td>
<td></td>
</tr>
<tr>
<td>6.21</td>
<td>291</td>
<td></td>
</tr>
<tr>
<td>6.22</td>
<td>291</td>
<td></td>
</tr>
<tr>
<td>6.23</td>
<td>292</td>
<td></td>
</tr>
<tr>
<td>6.24</td>
<td>295</td>
<td></td>
</tr>
<tr>
<td>6.25</td>
<td>295</td>
<td></td>
</tr>
<tr>
<td>6.26</td>
<td>295</td>
<td></td>
</tr>
<tr>
<td>6.27</td>
<td>295</td>
<td></td>
</tr>
<tr>
<td>6.28</td>
<td>298</td>
<td></td>
</tr>
<tr>
<td>6.29</td>
<td>299</td>
<td></td>
</tr>
<tr>
<td>6.30</td>
<td>301</td>
<td></td>
</tr>
<tr>
<td>6.31</td>
<td>302</td>
<td></td>
</tr>
<tr>
<td>7.1</td>
<td>313</td>
<td></td>
</tr>
<tr>
<td>7.2</td>
<td>314</td>
<td></td>
</tr>
<tr>
<td>7.3</td>
<td>316</td>
<td></td>
</tr>
<tr>
<td>7.4</td>
<td>316</td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>319</td>
<td></td>
</tr>
<tr>
<td>7.6</td>
<td>321</td>
<td></td>
</tr>
<tr>
<td>7.7</td>
<td>321</td>
<td></td>
</tr>
<tr>
<td>7.8</td>
<td>324</td>
<td></td>
</tr>
<tr>
<td>7.9</td>
<td>324</td>
<td></td>
</tr>
<tr>
<td>7.10</td>
<td>324</td>
<td></td>
</tr>
<tr>
<td>7.11</td>
<td>324</td>
<td></td>
</tr>
<tr>
<td>7.12</td>
<td>327</td>
<td></td>
</tr>
<tr>
<td>7.13</td>
<td>327</td>
<td></td>
</tr>
<tr>
<td>7.14</td>
<td>327</td>
<td></td>
</tr>
<tr>
<td>7.15</td>
<td>327</td>
<td></td>
</tr>
<tr>
<td>7.16</td>
<td>330</td>
<td></td>
</tr>
</tbody>
</table>
7.17 Illustration of staining processes on the discontinuity wall in section A............ 330
7.18 Probability density histogram of discontinuity infilling materials in section A ........ 331
7.19 Frequency distribution of discontinuity infilling materials in section B................. 331
7.20 Probability density histogram of discontinuity roughness in section A................... 331
7.21 Probability density histogram of discontinuity roughness in section B................... 331
7.22 Frequency distribution of discontinuity persistence in section A.......................... 333
7.23 Probability density histogram of discontinuity persistence in section B.................. 333
7.24 Probability density histogram of discontinuity lengths in section A....................... 333
7.25 Frequency distribution of discontinuity lengths in section B.............................. 333
7.26 Frequency distribution of discontinuity spacings in section A............................. 335
7.27 Probability density histogram of discontinuity spacings in section B..................... 335
7.28 Probability density histogram of discontinuity spacings in Dunmore Quarry.......... 336
7.29 General view of a Heavily jointed basalt in Dunmore Quarry............................. 338
7.30 Measuring vertical discontinuities using cherry picker in Dunmore Quarry............. 344
7.31 Analysis of discontinuity orientation data of section A from Dunmore
Quarry ........................................................................................................... 345
7.32 Analysis of pole plot data of section A from Dunmore Quarry.............................. 346
7.33 Illustration of discontinuity data of section B from Dunmore Quarry.................... 348
7.34 Analysis of cluster poles of section B from Dunmore Quarry.............................. 349
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table No.</th>
<th>Page No</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1 Raw material extracted from New South Wales hard stone quarries</td>
<td>6</td>
</tr>
<tr>
<td>1.2 Structure of rock outcrops at New South Wales quarries</td>
<td>7</td>
</tr>
<tr>
<td>1.3 Rock type at New South Wales reduction sites on quarries</td>
<td>7</td>
</tr>
<tr>
<td>1.4 Summary of information on quarries in eastern Australia</td>
<td>8</td>
</tr>
<tr>
<td>1.5 Schedule of the research</td>
<td>20</td>
</tr>
<tr>
<td>2.1 Description of number of joint sets</td>
<td>31</td>
</tr>
<tr>
<td>2.2 Description of divisions of aperture</td>
<td>34</td>
</tr>
<tr>
<td>2.3 Description of joint spacing</td>
<td>39</td>
</tr>
<tr>
<td>2.4 Description of discontinuity persistence</td>
<td>40</td>
</tr>
<tr>
<td>2.5 Description of block size</td>
<td>43</td>
</tr>
<tr>
<td>2.6 Description of rock weathering classifications</td>
<td>52</td>
</tr>
<tr>
<td>2.7 Engineering classification of intact rock</td>
<td>61</td>
</tr>
<tr>
<td>2.8 Major engineering rock mass classification currently in use</td>
<td>64</td>
</tr>
<tr>
<td>2.9 Various ranges of (RQD) rock quality designation</td>
<td>66</td>
</tr>
<tr>
<td>2.10 Geomechanics classification system</td>
<td>70</td>
</tr>
<tr>
<td>2.11 Illustration of RMR classes</td>
<td>71</td>
</tr>
<tr>
<td>2.12 Weakening Coefficient classification system (WC)</td>
<td>75</td>
</tr>
<tr>
<td>3.1 Dimensions of NX samples for different tests</td>
<td>83</td>
</tr>
<tr>
<td>3.2 Specification of the Z. J drilling machine</td>
<td>86</td>
</tr>
<tr>
<td>3.3 Results of water content and density tests</td>
<td>90</td>
</tr>
<tr>
<td>3.4 The main specification of Schenck Trebel testing machine</td>
<td>96</td>
</tr>
<tr>
<td>3.5 Mechanical properties of rocks resulted from uniaxial tests</td>
<td>102</td>
</tr>
<tr>
<td>3.6 Results of measured tensile strength, specimen from sections A and B</td>
<td>120</td>
</tr>
<tr>
<td>of Mugga II Quarry</td>
<td></td>
</tr>
<tr>
<td>3.7 Results of measured tensile strength, specimen from sections C and D</td>
<td>121</td>
</tr>
<tr>
<td>of Mugga II Quarry</td>
<td></td>
</tr>
<tr>
<td>Table No.</td>
<td>Page No</td>
</tr>
<tr>
<td>----------</td>
<td>---------</td>
</tr>
<tr>
<td>3.8</td>
<td>Results of measured tensile strength, specimen from Marulan Quarry</td>
</tr>
<tr>
<td>3.9</td>
<td>Results of measured tensile strength, specimen from Dunmore Quarry</td>
</tr>
<tr>
<td>3.10</td>
<td>Results of axial point load index test, specimen from sections A and B of Mugga II Quarry</td>
</tr>
<tr>
<td>3.11</td>
<td>Results of diametral point load index test, specimen from sections A and B of Mugga II Quarry</td>
</tr>
<tr>
<td>3.12</td>
<td>Results of axial point load index test, specimen from sections C and D of Mugga II Quarry</td>
</tr>
<tr>
<td>3.13</td>
<td>Results of diametral point load index test, specimen from sections C and D of Mugga II Quarry</td>
</tr>
<tr>
<td>3.14</td>
<td>Results of axial point load index test, specimen from Marulan Quarry</td>
</tr>
<tr>
<td>3.15</td>
<td>Results of diametral point load index test, specimen from Marulan Quarry</td>
</tr>
<tr>
<td>3.16</td>
<td>Results of axial point load index test, specimen from Marulan Quarry</td>
</tr>
<tr>
<td>3.17</td>
<td>Results of diametral point load index test, specimen from Marulan Quarry</td>
</tr>
<tr>
<td>4.1</td>
<td>Experimental values of basic friction angle for various rocks</td>
</tr>
<tr>
<td>4.2</td>
<td>Different values of joint compressive strength (JCS) for different type of hard rocks</td>
</tr>
<tr>
<td>4.3</td>
<td>Roughness values for the different sections of quarries</td>
</tr>
<tr>
<td>4.4</td>
<td>The shear strength parameters for natural jointed samples from different quarries</td>
</tr>
<tr>
<td>5.1</td>
<td>Results of descriptive statistical analysis of discontinuity orientations</td>
</tr>
<tr>
<td>5.2</td>
<td>Frequency distribution of discontinuity dip angle in Mugga II Quarry</td>
</tr>
<tr>
<td>5.3</td>
<td>Statistical analysis of discontinuity water condition in Mugga II Quarry</td>
</tr>
<tr>
<td>5.4</td>
<td>Frequency distribution of discontinuity curvature in Mugga II Quarry</td>
</tr>
<tr>
<td>5.5</td>
<td>Frequency distribution of discontinuity persistence in Mugga II Quarry</td>
</tr>
<tr>
<td>5.6</td>
<td>Summary of field data and negative exponential curve parameters</td>
</tr>
<tr>
<td>5.7</td>
<td>Frequency distribution of discontinuity spacings in Mugga II Quarry</td>
</tr>
<tr>
<td>5.8</td>
<td>Comparison between measured and theoretical RQD in a porphyry rock mass</td>
</tr>
<tr>
<td>Table No.</td>
<td>Page No</td>
</tr>
<tr>
<td>------------------------------------------------------------------------</td>
<td>---------</td>
</tr>
<tr>
<td>5.9 Different values of RQD, RQP, and RQR for a porphyry rock mass</td>
<td>235</td>
</tr>
<tr>
<td>5.10 Trace length distribution</td>
<td>239</td>
</tr>
<tr>
<td>5.11 Results of frequency distribution of discontinuity lengths in</td>
<td>241</td>
</tr>
<tr>
<td>Mugga II Quarry</td>
<td></td>
</tr>
<tr>
<td>5.12 Factor loading of the stability evaluation from the contributory factors</td>
<td>242</td>
</tr>
<tr>
<td>5.13 Orientation of major discontinuity planes and slope faces</td>
<td>256</td>
</tr>
<tr>
<td>5.14 Engineering properties of porphyry intact rock samples from</td>
<td>261</td>
</tr>
<tr>
<td>Mugga II Quarry</td>
<td></td>
</tr>
<tr>
<td>5.15 Results of geomechanics classification system (RMR) in Mugga II Quarry</td>
<td>264</td>
</tr>
<tr>
<td>5.16 Results of Q system in Mugga II Quarry</td>
<td>265</td>
</tr>
<tr>
<td>5.17 Results of weakening coefficient classification (WC) in</td>
<td>266</td>
</tr>
<tr>
<td>Mugga II Quarry</td>
<td></td>
</tr>
<tr>
<td>5.18 Engineering description of porphyry rock mass in Mugga II Quarry</td>
<td>268</td>
</tr>
<tr>
<td>6.1 Results of descriptive statistical analysis of discontinuity orientations data from Marulan Quarry</td>
<td>282</td>
</tr>
<tr>
<td>6.2 Frequency distribution of discontinuity dip angle in Marulan Quarry</td>
<td>283</td>
</tr>
<tr>
<td>6.3 Results of statistical analysis of discontinuity water condition</td>
<td>284</td>
</tr>
<tr>
<td>6.4 Results of statistical analysis of discontinuity curvatures</td>
<td>287</td>
</tr>
<tr>
<td>6.5 Results of statistical analysis of discontinuity persistence</td>
<td>289</td>
</tr>
<tr>
<td>6.6 Frequency distribution of discontinuity spacings in Marulan Quarry</td>
<td>290</td>
</tr>
<tr>
<td>6.7 Summary of field data and negative exponential curve parameters</td>
<td>293</td>
</tr>
<tr>
<td>6.8 A comparison between measured and theoretical RQD</td>
<td>293</td>
</tr>
<tr>
<td>6.9 Different Valued of (RQD), (RQP), and (RQR) for a limestone quarry</td>
<td>294</td>
</tr>
<tr>
<td>6.10 Results of frequency distribution of discontinuity lengths in</td>
<td>296</td>
</tr>
<tr>
<td>Marulan Quarry</td>
<td></td>
</tr>
<tr>
<td>6.11 Orientation of major discontinuity planes and slope faces</td>
<td>304</td>
</tr>
<tr>
<td>6.12 Engineering properties of intact rock samples from Marulan Quarry</td>
<td>305</td>
</tr>
<tr>
<td>6.13 Results of geomechnics classification system in Marulan Quarry</td>
<td>306</td>
</tr>
<tr>
<td>6.14 Results of Q system in Marulan Quarry</td>
<td>307</td>
</tr>
<tr>
<td>6.15 Results of weakening coefficient classification in Marulan Quarry</td>
<td>308</td>
</tr>
<tr>
<td>6.16 Engineering description of limestone in Marulan Quarry</td>
<td>309</td>
</tr>
</tbody>
</table>
7.1 Representation of probability density distribution of discontinuity dip angles in Dunmore Quarry

7.2 Results of descriptive statistical analysis of discontinuity orientations data from Dunmore Quarry

7.3 Results of descriptive statistical analysis of discontinuity water condition

7.4 Results of descriptive statistical analysis of discontinuity curvatures in Dunmore Quarry

7.5 Results of descriptive statistical analysis of discontinuity persistence in a basalt rock mass

7.6 Results of descriptive statistical analysis of discontinuity lengths data in Dunmore Quarry

7.7 Summary of field data and negative exponential curve parameters

7.8 Results of descriptive statistical analysis of discontinuity spacings data in Dunmore Quarry

7.9 Comparison between measured and theoretical RQD in Dunmore Quarry

7.10 Different values of (RQD), (RQP), and (RQR) for a basalt rock mass

7.11 Correlation matrix of discontinuity parameters in the slope stability analysis of Dunmore Quarry

7.12 Factor analysis of the stability evaluation from the contributor factors

7.13 Orientations of major discontinuity planes and slope faces in Dunmore Quarry

7.14 Engineering properties of intact rock samples from Dunmore Quarry

7.15 Results of RMR classification in Dunmore Quarry

7.16 New rating proposed for the RMR classification system

7.17 Results of engineering rock mass quality (Q system) in Dunmore Quarry

7.18 Results of weakening coefficient (WC) system in Dunmore Quarry

7.19 Engineering description of basalt rock mass in Dunmore Quarry
LIST OF SYMBOLS AND ABBREVIATIONS

A : area (mm$^2$)
A : constant parameter in the power law failure criterion
A : function of rock mass quality
ACT : Australian Capital Territory
AD : area of joint plane
ad$i$ : area of the $i$th joint
as : shear area ratio
B : constant parameter in the power law failure criterion
C : cohesion
CNL : constant normal load
CNS : constant normal stiffness
CT : computerised tomography
Cj : joint cohesion
Co : uniaxial compressive strength
D : diameter of sample
d$\text{m}$ : most common distance measured
DRMS : design rock mass strength
$E_{(50)}$ : Young’s modulus
$E_M$ : in-situ rock mass modulus (GPa)
F : factor of safety
i : angle of inclination of the teeth
I$a$ : strength anisotropy index
I$b$ : block size index
I$s$ : point load index
I$sa$ : axial point load index
I$sd$ : diametral point load index
ISRM : international society for rock mechanics
Ja : joint alteration degree
JCS : joint compressive strength
JRC : Joint roughness coefficient
Jn : joint set number
Jr : rating of joint roughness
Jr : volumetric joint count
JRC : joint roughness coefficient
Jw : joint water reduction number
K : dilation rate (when as = 0 and σ = 0), page 165
K : triaxial stress factor (page 109)
K : joint persistence index (page 223)
K : stiffness
K -Ar : Potassium Argon dating method
K1 : discontinuity aperture
K2 : joint spacing index
K3 : joint surface index
K4 : joint infilling material index
L : length of sample
LEFM : linear elastic fracture mechanics
Lₙ : the point of forces acting against the toppling of nth block
Lₛ : length of a straight segment S
Lₘᵢ : length of the ith joint segment in S
m : constant for rock mass in the equations of Hoek and Brown and of Bieniawski
m₁ : mass of container and the moist rock sample
m₂ : mass of container and the dried rock sample
mᵢ : constant for intact rock in the Hoek and Brown criterion
Mₙ : the point of forces acting on the toppling of nth block
MRMR : mining rock mass rating
Ms : mass of solid particles
Msat : saturated sample
Mw : mass of water
N : north
Nφ : friction number
NGI : Norwegian Geotechnical Institute
NSW : New South Wales
P : load (kN)
Pₙ : the force acting on the toppling of nth block
Pₙ₋₁ : the force acting the forces on the toppling of block (n-1)
Q : rock mass quality index
R : resistance force
R : rock quality designation
RA : normal reaction by plane A
RAP : rating adjustment parameters
RB : normal reaction by plane B
RBR : rock bridge rupture
Rj : shear resistance of rock mass
RMR : rock mass rating
RMS : rock mass strength
RQD : rock quality designation
RQP : rock quality percentage
RQR : rock quality risk
Rr : shear resistance of intact rock
RSR : rock structure rating
s : constant for rock mass in the Hoek and Brown criterion
S : spacing
F : safety factor
si : constant for intact rock in the Hoek and Brown criterion
SRF : stress reduction factor
Std Div : standard deviation
T500 : strength index (for a sample with 500 mm² cross-section)
Ti : termination index
U : uplift force
UCS : uniaxial compressive strength
v : dilation rate
V : force of water
Vp : dilation rate at peak shear strength
W : weight of block
WC : weakening coefficient
X : tangential distance along the profile
Y : amplitude at a particular point on the roughness profile
Yn : the height of the block
α : strike
\( \alpha \) : the angle between the surface of toppling with the horizon
\( \beta \) : dip
\( \beta' \) : the angle between rock bolt and sliding face
\( \Delta x \) : horizontal displacement
\( \Delta x_n \) : width of the nth block
\( \Delta z \) : vertical displacement
\( \Delta u \) : shear displacement
\( \varepsilon_a \) : axial strain
\( \varepsilon_d \) : diametral strain
\( \varepsilon_v \) : volumetric strain
\( \phi \) : internal friction angle
\( \phi' \) : effective friction angle
\( \phi_b \) : basic friction resistance
\( \phi_f \) : average value of friction angle
\( \phi_j \) : joint friction angle
\( \phi_r \) : residual friction angle
\( \phi_s \) : surface roughness coefficient
\( \phi_u \) : angle of friction sliding resistance
\( \bar{l} \) : mean discontinuity trace length
\( \lambda \) : average number of discontinuity per metre
\( \mu \) : mean trace termination frequency
\( \rho \) : bulk density
\( \rho_d \) : dry density
\( \rho_{sat} \) : saturated density
\( \theta \) : Poisson’s ratio
\( \sigma \) : stress
\( \sigma'_n \) : effective normal stress
\( \sigma_1 \) : major principal stress at failure
\( \sigma_3 \) : confining pressure (minimum principal stress)
\( \sigma_c \) : uniaxial compressive strength
\( \sigma_{cf} \) : uniaxial compressive strength of joint
\( \sigma_{cm} \) : uniaxial compressive strength of the rock mass
\( \sigma_j \) : joint compressive strength

xxvi
\( \sigma_n \) : normal stress
\( \sigma_t \) : tensile strength
\( \tau \) : peak shear strength
\( \tau_D \) : shear strength due to dilation
\( \tau_{mb} \) : shear strength mobilised along the failure surface
\( \tau_p \) : shearing stress
\( \tau_r \) : shear strength of the rock materials
\( \bar{X} \) : mean discontinuity spacing
\( \psi \) : angle of slope face
Chapter 1
Rock Slope Stability Problems in Hard Rock Quarries
CHAPTER ONE

ROCK SLOPE STABILITY PROBLEMS IN HARD ROCK QUARRIES

1.1 INTRODUCTION

Engineering rock mechanics is concerned with solving complex problems regarding the behaviour of rock masses. Rock, as a material, is heterogeneous and the design of rock mass constructions requires a comprehensive knowledge of the presence of specific geological structures and the properties of rock mass discontinuities which are important for the stability analysis of rock slopes. The determination of reliable shear strength values is a critical component of slope design, because relatively small changes in shear strength can result in significant changes in the safe height or angle of a slope. The internal friction angle, measured from testing of naturally jointed samples, is necessary for the stability assessment of slope faces. The main purpose of slope stability analysis is to achieve a safer and economic design of rock slopes in surface mining operations.

Surface mining is one of the most commonly used methods for mining shallow ore deposits. It is a relatively low cost extraction method for the extraction of stones from quarries together with thin layer mineral deposits which are not economically extracted by underground techniques. In general, the objective of any mining operation is to extract ore from the ground and transport it to the processing plant at minimum cost.
As a result, improvements in surface mining technology permit many deposits which have previously been considered unprofitable for development as economic mining operations.

Over the last three decades increasing civil and mining engineering activities, particularly in developing countries, have brought about increased challenges for geotechnical engineers to design slopes around mining excavations. This has resulted in an increased understanding of new concepts associated with the stability of rock slopes in surface mines under the difficult geological situations which influence surface mining operations. Increasing demands for higher productivity and economy in surface mining operations has promoted the development of a greater understanding of geological and geotechnical problems.

The stability of rock slopes is predominantly affected by the physical and mechanical properties of rock mass discontinuities, through the frequency, orientation and shear strength parameters of the discontinuities. An understanding of the relationship between geological and geotechnical conditions, and the failure and displacement behaviour of rock slopes is crucial to slope stability analysis.

1.2 CLASSIFICATION OF SURFACE MINING METHODS

Exploitation, in which the mining of ore or stone is carried out at the surface with essentially no exposure of miners underground is referred to as surface mining. While openings may sometimes be placed below the surface and limited underground development work is occasionally required, this type of mining is essentially surface-based (Hartman, 1987). The method of mining determines the height of the high wall and spoil slopes, bench width, location and equipment specifications. The geometry of a high wall slope can be altered to improve its stability (Vakili and et al, 1991). The following mechanical methods are most commonly used in surface mining: (the following is based on Hartman, 1987).
1.2.1 Open Pit Mining

In open pit mining the overburden is removed (and transported to a disposal area) to uncover the mineral deposit. Open pit mining is a cost effective large-scale method in terms of production rate, and is responsible for 60% of all surface output.

1.2.2 Quarrying

Dimension-stone quarrying produces prismatic mineral blocks which are roughly sized and shaped. The quarries resemble open pits with benches which are lower in comparison with other open pit mining methods, and they are nearly vertical. In overall appearance, the wall of a quarry is often of imposing height and steepness; some attain a vertical dimension approaching 300 m.

Quarrying is now an uncommon mining method in the United States for the simple reason that relatively little dimension stone is produced today. It is common in some countries such as Australia, Spain and Italy. In the case of quarrying stone, due to the highly specialised nature of dimension stone quarrying, a customised cycle of unit operation is employed (Spielvogel, 1978), cited by Hartman, (1987).

1.2.3 Open Cast Mining

Open cast mining is a surface exploitation method, used mainly for soft coal extraction. In this method, the overburden is not transported to a waste dump for disposal but cast or hauled directly into adjacent mined-out strips. Hartman, (1987) states that one of the advantages of this method is that the cut is opened in a relatively short time, permitting a much steeper slope to the bank.

1.2.4 Auger Mining Method

Auger mining is another method of surface mining, mainly employed for the extraction of soft rock or coal by boring or excavating an opening into the seam beneath the
overburden. By this method, although the mineral is extracted by a mining machine working underground, the crew operating the machine is located at the ground surface. It should be noted that this method is only occasionally used in surface mining operations in cases where the ground is very soft and mining near the high walls.

The most important advantages of open pit mining methods are summarised by Hartman (1987). Open pit mining is cost effective, highly productive with good recovery (nearly 100%) and it has a good health and safety record in comparison with underground methods.

The most important disadvantages of open pit mining operations are that there are depth and stripping ratio limitations. They require large deposits to be cost effective, and have slope stability problems (design and problems with maintenance of the benches, drainage problems, failures, and rock falls). Quarrying and surface mining operations in Australia are briefly reviewed in the following paragraphs.

1.3 QUARRYING AND SURFACE MINING OPERATIONS IN AUSTRALIA

Quarries are generally accepted as surface excavations from which fairly massive and deep deposits of hard rock are extracted, usually for the production of aggregates. The excavations are fairly deep, and tend to work progressively outwards and downwards. The term 'pit' is used for fairly shallow excavations for extracting soft materials which often occur in bedded or clearly defined deposits.

Surface mining is the most commonly used method for recovering mineral resources, particularly from coal mines and quarries in Australia. Quarrying is arguably Australia's fourth most important industry. Quarries produce a range of products, including various sizes of aggregate, fine crushed rock, rail ballast, gabion, asphalt aggregate, as well as prismatic blocks of minerals.

The quarries resemble open pits, but have benches which are lower and nearly vertical (Hartman, 1987). From the operational point of view, pits and quarries have been divided into four distinct types by Coppin and Bradshaw, (1982) as follows:
(a) **Shallow Pits with Little Overburden**

Shallow pits with little overburden are mainly associated with soft recently deposited materials, such as sand, chalk or clay. The nature of the pit floor and the sort of fill available nearby for restorative works will determine the potential value of the material.

(b) **Shallow Pits with a Thick Overburden or Spoil**

Shallow pits with a thick overburden or spoil are very variable, depending on the particular circumstances and nature of the deposits. A classic strip-mine sequence is usually followed, where overburden layers are removed with a dragline, or by a bucket wheel excavator and conveyor belt and then dumped back into a previously worked cut.

(c) **Deep Quarries or Pits with Little Overburden or Spoil, Either Flooded or Dry**

Deep quarries or pits with little overburden or spoil, either flooded or dry are typical of the crushed rock or chalk industry. Backfilling of the excavation is not viable during the life of the quarry, because this leads to the sterilisation of reserves.

(d) **Deep Quarries or Pits with a Fairly Thick Overburden or Spoil**

Deep quarries or pits with a fairly thick overburden or spoil occur in mineral deposits that have a fairly high value, such as china clay, coal and slate, where it is economic to work with much higher waste to a product ratio of 7:1 for china clay and 20:1 for slate. Because these quarries work progressively outwards and downwards, all the spoil has to be disposed of away from the site without sterilizing future reserves. It should be noted that problems of waste disposal in these excavations are considerable. The study areas of this research are situated in New South Wales, Australia. In the following sections, the distribution and activities of the quarries in NSW are reviewed.
1.4 REVIEW OF THE SITES IN NEW SOUTH WALES

One hundred and eighty three quarry sites have been recorded in the New South Wales sites register. Of the quarries, one hundred and forty four were considered to be hard stone quarries while seventeen were ochre quarries. All hard stone quarries in NSW were sources of rock with quarry/pits. The type of rock which had been extracted was recorded for one hundred and four (72.2 %) of the one hundred and forty four hard stone quarry sites. The range and frequency of rock types quarried are presented in Table 1.1. The nature of the rock outcrop was recorded for 41 hard rock quarries (Table 1.2). Rock type was recorded for 12 of these sites and, in nearly all cases, is volcanic or metamorphic in origin. Table 1.3 shows different rock types at NSW reduction sites on quarries.

Table 1.1 Raw material extracted from New South Wales hard stone quarries (After Hiscock & Mitchell, 1993).

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Number</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silcrete</td>
<td>28</td>
<td>19.4</td>
</tr>
<tr>
<td>Chert</td>
<td>16</td>
<td>11.1</td>
</tr>
<tr>
<td>Quartz</td>
<td>15</td>
<td>10.4</td>
</tr>
<tr>
<td>Quartzite</td>
<td>12</td>
<td>8.3</td>
</tr>
<tr>
<td>Basalt</td>
<td>9</td>
<td>6.3</td>
</tr>
<tr>
<td>Sandstone</td>
<td>3</td>
<td>2.1</td>
</tr>
<tr>
<td>Greywacke</td>
<td>3</td>
<td>2.1</td>
</tr>
<tr>
<td>Andesite</td>
<td>3</td>
<td>2.1</td>
</tr>
<tr>
<td>Porphyry</td>
<td>3</td>
<td>2.1</td>
</tr>
<tr>
<td>Granite</td>
<td>2</td>
<td>1.4</td>
</tr>
<tr>
<td>Mudstone</td>
<td>2</td>
<td>1.4</td>
</tr>
<tr>
<td>Siltstone</td>
<td>2</td>
<td>1.4</td>
</tr>
<tr>
<td>Chalcedony</td>
<td>1</td>
<td>0.7</td>
</tr>
<tr>
<td>Ironstone</td>
<td>1</td>
<td>0.7</td>
</tr>
<tr>
<td>Amphibolite</td>
<td>1</td>
<td>0.7</td>
</tr>
<tr>
<td>Other volcanic</td>
<td>3</td>
<td>2.1</td>
</tr>
<tr>
<td>Unknown</td>
<td>40</td>
<td>27.8</td>
</tr>
<tr>
<td>Total</td>
<td>144</td>
<td>100</td>
</tr>
</tbody>
</table>
The nature of the rock outcrop was recorded for 41 hard stone quarries. These data are presented in Table 1.2.

Table 1.2 Structure of rock outcrops at New South Wales quarries
(After Hiscock & Mitchell, 1993).

<table>
<thead>
<tr>
<th>Structure</th>
<th>Number</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulders</td>
<td>21</td>
<td>51.2</td>
</tr>
<tr>
<td>Pebbles/cobbles</td>
<td>11</td>
<td>26.8</td>
</tr>
<tr>
<td>Exposed vein or block</td>
<td>6</td>
<td>14.6</td>
</tr>
<tr>
<td>Gibber</td>
<td>2</td>
<td>4.8</td>
</tr>
<tr>
<td>Gravel</td>
<td>1</td>
<td>2.5</td>
</tr>
<tr>
<td>Total</td>
<td>41</td>
<td>99.9</td>
</tr>
</tbody>
</table>

The rock type was recorded for twelve sites and, in nearly all cases, is volcanic or metamorphic in origin. Different type of rocks in NSW quarries are summarised in Table 1.3.

Table 1.3 Rock type at New South Wales reduction sites on quarries
(After Hiscock & Mitchell, 1993).

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Number</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basalt</td>
<td>4</td>
<td>33.3</td>
</tr>
<tr>
<td>Andesite</td>
<td>2</td>
<td>16.6</td>
</tr>
<tr>
<td>Porphyry</td>
<td>2</td>
<td>16.6</td>
</tr>
<tr>
<td>Amphibolite</td>
<td>1</td>
<td>8.3</td>
</tr>
<tr>
<td>Metavolcanic</td>
<td>1</td>
<td>8.3</td>
</tr>
<tr>
<td>Other Volcanic</td>
<td>1</td>
<td>8.3</td>
</tr>
<tr>
<td>Greywacke</td>
<td>1</td>
<td>8.3</td>
</tr>
<tr>
<td>Total</td>
<td>12</td>
<td>99.9</td>
</tr>
</tbody>
</table>
The data from Queensland and Victoria is given in Table 1.4. Many of the patterns which occur in New South Wales are repeated in the other two states.

Table 1.4 Summary of information on quarries in eastern Australia (After Hiscock & Mitchell, 1993).

<table>
<thead>
<tr>
<th>Information</th>
<th>Queensland</th>
<th>NSW</th>
<th>Victoria</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of quarries</td>
<td>156</td>
<td>183</td>
<td>65</td>
<td>404</td>
</tr>
<tr>
<td>Percentage with no information</td>
<td>23.2</td>
<td>10.4</td>
<td>21.5</td>
<td>17.1</td>
</tr>
<tr>
<td>Percentage with information about mode of extraction</td>
<td>28.3</td>
<td>10.9</td>
<td>63.1</td>
<td>26.0</td>
</tr>
<tr>
<td>Percentage with information on area of quarrying</td>
<td>3.5</td>
<td>5.5</td>
<td>6.2</td>
<td>5.0</td>
</tr>
<tr>
<td>Percentage with information on structure of quarried stone material</td>
<td>26.5</td>
<td>25.1</td>
<td>40.0</td>
<td>28.0</td>
</tr>
<tr>
<td>Percentage of quarries with associated reduction site</td>
<td>100</td>
<td>99.3</td>
<td>92.3</td>
<td>98.5</td>
</tr>
<tr>
<td>Percentage of Hard stone quarries with surficial extraction</td>
<td>75.0</td>
<td>60.0</td>
<td>76.9</td>
<td>68.6</td>
</tr>
<tr>
<td>Number of ochre quarries</td>
<td>3</td>
<td>17</td>
<td>2</td>
<td>22</td>
</tr>
<tr>
<td>Ochre quarries as a percentage of all quarries</td>
<td>1.9</td>
<td>9.3</td>
<td>3.1</td>
<td>5.5</td>
</tr>
<tr>
<td>Number of reduction sites</td>
<td>252</td>
<td>2010</td>
<td>87</td>
<td>2349</td>
</tr>
<tr>
<td>Percentage away from quarries</td>
<td>55.2</td>
<td>90.1</td>
<td>32.2</td>
<td>84.2</td>
</tr>
<tr>
<td>Grinding groove as % of reduction sites away from quarries</td>
<td>89.7</td>
<td>100.0</td>
<td>100.0</td>
<td>99.3</td>
</tr>
</tbody>
</table>

1.5 TYPICAL MODES OF FAILURE ON SOFT AND HARD ROCKS

Rocks typically contain many structural weaknesses which significantly reduce the shear strength of the mass below that of the intact material. Structural weaknesses also have a controlling influence on the movement of groundwater through the rock masses.
Stability analysis follows the typical failure modes of different soil and rock masses. One of the most important aspects of rock slope stability is the systematic collection and presentation of geological data that it can be readily evaluated and incorporated into stability analysis. The stability of slopes falls naturally into three division; the behaviour of soils or soft rocks, where stability is critical as in road and trail cutting; the behaviour of soft rocks in bedded deposit open cut mining; and finally the behaviour of hard rock in deep open pits (Thomas, 1985).

In mining excavations the slopes do not always have to possess permanent stability, and in many cases a lower factor of safety may have to be adopted to produce economic working. Thomas, (1985) stated that “In a large conical shaped pit, for every degree, the pit wall slope is flattened, then several million dollars may be spent in removing the extra overburden”. It should be noted that various techniques are available to assess the stability of hard rock slopes. Between the current techniques, the stereographical projection provides a conventional method for the presentation of geological data. The use of stereographical projection, commonly the Schmidt net, means that the trace of planes on the surface of the ‘reference sphere’ can be used to define the dips and dip directions of the planes. In the following paragraphs the stability of soil and soft rock slopes as well as the stability of hard rock slopes are outlined.

1.5.1 Soil and Soft Rock Slopes

The full theory and practice of slope design in soil and soft rock can be found in standard text books on soil mechanics and geotechnical engineering. Rocks can be considered as weak in the sense that they consist of low strength intact materials or due to pervasive fracturing the rock mass behaviour in a weak manner. The study of weak rocks for engineering purposes has received a considerable attention over the last three decades. The rock or soil has a shear strength which enable it to stay in position on an inclined plane. The shear strength is normally expressed by the Coulomb’s equation;

\[ \tau = C + \sigma_n \tan \phi \]  

(1.1)
where

\[ \tau \] is the shear strength.

\[ C \] is the cohesion along the failure plane, or the shear strength under zero normal stress.

\[ \sigma_n \] is the normal stress to the failure plane.

\[ \phi \] is the angle of friction of the material along the failure surface.

It should be noted that the values of \( \tau \), \( C \), and \( \phi \) can be determined easily by field or laboratory tests using a shear box apparatus.

To extend the Coulomb's equation into rock masses, it is necessary to make into account the effect of rock joints and roughness. The effect of surface roughness can be expressed by adding an extra value \( (\phi_r) \) to the original friction angle, (Thomas, 1985)

\[ \tau = C + \sigma_n \tan (\phi + \phi_r) \]  \hspace{1cm} (1.2)

Resistance of sliding depends not only on friction angle but also on the normal stress across the sliding surfaces.

The types of failure and methods of stability assessment of soils and soft rocks are different from hard rocks. Stability analysis of soft rocks and soil formations is normally run on various types of failure modes such as, circular, non circular, plain and wedge failures. In weak rocks or soil, the passing of heavy trucks results in an increase in pore water pressure due to the compaction of underlying formations; while in a rock slope it can cause sliding of some blocks from the benches of open pit mine.

### 1.5.2 Hard Rock Slopes

Most of open pit mines that are deep enough to be concerned with the stability assessment are extracting from the jointed rock masses. The overall slope angle of open pit mines are usually about 45 degrees while individual benches have a slope of 55° to 80° from the horizontal and between 15 to 20 m high. The reason for these variations is that rock masses contain geological structures and discontinuities.
Hard rock mass are liable to sudden and violent failure if their peak strength is exceeded by excavating and extremely steep or high slope. On the other hand soft materials, which show small differences between peak and residual strength, tend to fail by gradual sliding.

According to the Hoek and Bray, (1981) the principal types of failure which are generated in a hard rock mass are plane, toppling and wedge failures. Plane failure occur in highly ordered structure such as slate. Toppling failure is generally associated with steep slopes in which the jointing is near vertical. This type of failure also takes place in hard rocks when columnar structures are separated by steeply dipping discontinuity. Wedge failure is a type of translation failure in which two planar discontinuities intersect, the wedge so formed daylighting into the face. In other words failure may occur if the line of intersection of both planes dips into the slope at an angle less than that of the slope.

It should be noted that this study focuses on the stability of hard rocks, particularly in quarries and open pit mines in Australia. In this regard, three different type of quarries were studied from the point of view of hard rock slope stability problems. These quarries are, Mugga II Quarry, Marulan Quarry, and Dunmore Quarry.

In the following paragraphs most important factors affecting rock slope stability are presented. The factors which influence the instability of hard rock slopes can be divided into two categories (internal and external factors) which are briefly reviewed in the following paragraphs.

1.6 FACTORS AFFECTING THE STABILITY OF ROCK SLOPES

Extensive studies into the field of rock slope stability has been carried out by many researchers including Hoek & Bray (1977), Hoek & Brown (1980), Priest, (1980), Brawner (1975, 1982), Hoek (1974, 1982), Brackley et al (1989), Sakurai (1993) etc. Research has shown that several important factors influence slope stability.
In designing large excavations for open-pit mines, selection of the slope angle is the most important decision for engineers. The purpose of applying geotechnical studies to the slope is to ensure the greatest economy together with reasonable stability. Because the rock mass of each high wall slope is unique, there are no standard solutions for slope stability analysis (Vakili and et al, 1991). The study areas are presented in Figure 1.1.

1.6.1 The Influence of Internal Factors on Rock Slope Stability

Internal factors include the inherent properties of the rock itself, and the external factors are environmental (which has an important influence). Internal factors include the mineralogical composition and texture, as well as the density and porosity of rocks. Apart from the frequency of joints, the orientation of joints with respect to the loading
direction assumes greater significance from the point of view of stability (Ramamurthy and Arora, 1994).

The most important intrinsic factors affecting rock slope stability include the following:

- Properties of intact rock material (strength, porosity, density, water content).
- Lithological composition or the minerals constituting the rocks.
- Properties of discontinuities (aperture, roughness, and filling material).
- Groundwater flow and pore water pressure.
- The range and extent of discontinuities.
- The orientation of discontinuities.
- Numbers of joint sets.

1.6.2 The Influence of the External Factors on Rock Slope Stability

The effect of external loads on rock slope stability is related to the equipment used, such as draglines, heavy trucks, shovels, the weight of the block and the dynamic loads as well as pore pressure due to the presence of water. A consideration of rock slope stability shows that static surcharge loads have a minor effect on instability and resulting rock slides in comparison with dynamic loads produced by earthquake and vibration. Some equipment such as heavy trucks and compressors can have a significant effect on the stability of the slopes. In addition waste material may increase external loading on the critical potential failure surfaces.

The most important external factors are as follows:

- **Operational factors:**
  (a) Drilling
  (b) Blasting
  (c) Weight of equipment:
(I) Dragline  
(ii) Heavy trucks  
(iii) Shovels  
(d) Time  

- **Geometrical factors**  
  (a) Slope angle  
  (b) Slope height  
  (c) Width of the benches  
  (d) Overall slope  
  (e) Weight of block  

- **Environmental factors**  
  (a) Weight of water  
  (b) Earthquake  

1.6.3 The Influence of Vibration on the Stability of Hard Rock Slopes  

It is interesting to consider the effects of vibration due to blasting. Blasting is one of the main surface mining operations carried out in open pit mines to extract rock. The most important aims of blasting can be divided into three categories:  

- Fragmentation of rock mass into small blocks.  
- Optimisation in terms of cost of recovery and accuracy of excavation.  
- Safety of the operation, with regard to the human, and mining equipment.  

Blasting has a considerable influence on the sliding of slopes, particularly when the materials of rock masses are very jointed.  

Ladegoard-Pedersen and Dally (1975) identified four types of damage caused by blasting:  

- Structural damage due to vibrations induced in the rock mass.  
- Damage due to fly-rock or boulders ejected from the blast area.
• Damage due to airblast.

• Damage may also occur because of instability of slopes.

The potential for sliding may increase due to the effects of vibration. Because of this, the design of blasting methods should be carried out with a serious concern for safety. The influence of blasting on the stability of rock masses is reported by Brawner et al. (1986) in a number of ways:

• Shear stresses in the slopes are increased by seismic acceleration forces.

• Discontinuities are opened and new fractures are developed.

• Ground-water permeates open cracks increasing the rate of weathering.

• Excess blasting usually increases the occurrence of rock falls from benches.

• Shear strain along existing discontinuities frequently occurs as a result of a nearby blast.

1.6.4 The Influence of Drilling and Blasting on the Slope Stability of Hard Rocks

The aim of drilling in open-pit mining is to make a borehole for blasting purposes. Generally, in order to achieve the best results from blasting, the optimisation of drilling practice is essential. The effectiveness of drilling is highly dependent on the quality of the equipment and the skill of the operator. Vibration, due to drilling, can be a dominant factor influencing the potential for slope failure.

1.6.5 The Human Safety Problems Due to Open Pit Mining

The main purpose of slope analysis of hard rocks in open pit mining is to obtain a safe and economic design. The importance of slope stability analysis will therefore be obvious, particularly if a major instability occurs. Furthermore, the occurrence of slope failure of a significant scale can create considerable economic and personal damage. Therefore, in designing a slope for an open pit mine, many factors such as the safety of men and equipment should be considered or it may lead to non-profitable operations and loss of life and equipment.
1.6.6 Mine Operation and Economic Problems

Surface mine operations generally require drilling, blasting, loading and transportation processes which directly or indirectly influence the stability or instability of the slopes. The stability of the entire pit slope depends on different factors, as follows:

- Type of lithology
- Ground-water and surface water condition
- Number of benches
- Width of benches
- Depth of mining
- Height of benches
- Slope angle of mine.
- Geological structures
- Geotechnical parameters
- The stability of benches and their design also depends on the type of equipment being used and the daily production.

1.7 RESEARCH SIGNIFICANCE

During the last four decades comprehensive research into the field of rock engineering has been carried out by many authors such as; Brawner (1982), Hoek (1982), Hoek and Bray (1981) Brady & Brown (1993). The results of their work have shown that the stability of slopes in a jointed rock mass is an essential factor from the point of view of human safety and economy. Hence, open pit mining is one of the most widely applied methods of mining in Australia and most of the quarries and many coal mines operate by this method. In these circumstances, economic and safer operation are essential factors to establish the industry at a competitive cost.
The consideration of rock slope stability in surface mining and finding solutions to difficulties are the main responsibilities of mining engineers and geologists. Stabilisation and optimisation of hard rock slopes can reduce the human hazards and lead to saving the money during the life of the open pit mines. This research is an attempt to identify the most important problems regarding the stability of hard rock slopes and to introduce some remedial methods in order to reduce the potential of failure in open pit mining operations.

1.8 RESEARCH METHODOLOGY

This thesis is mainly based on empirical investigation. For this research, three different quarries were studied. In each site, the procedures involve the identification of mechanical and geotechnical characteristics of slope failures in conjunction with the field geology and discontinuity data analysis, as well as laboratory investigation on intact rock samples.

During site investigation, the main types of rocks and their structures were identified and discontinuities properties were studied. In order to study the mechanical behaviour of joints, several block samples containing natural joints were collected for testing in the Departmental Laboratory. From the block samples, several core specimen were prepared for laboratory testing.

Different laboratory tests were carried out in order to assess the engineering properties of the intact rock and rock mass. Laboratory testing results were used in rock mass classification and were also used in order to do some correlation between intact rock properties and the shear strength parameters. Computer analysis techniques (Graphical slope stability analysis method) were employed to evaluate the stability of slopes and determine the mode of slope failure. A slope stability flowchart is presented in Figure 1.2 for the purpose of stability assessment in a heavily jointed rock mass in surface mining investigations. As is clear from this flowchart, the main stages in conjunction with the stability assessment of hard
Slope Stability Assessment Flowchart in Jointed Rock for Surface Mining Purposes

1. Definition of problem
2. Selection of site
3. Site Investigation
4. Sampling
   - Geological data collection
   - Geotechnical data collection
5. Laboratory tests:
   - Shear test
   - Triaxial test
   - U.C.S. test
   - Point load test
   - Brazilian test
6. Data collection for slope stability analysis
7. Stability analysis
8. Remedial work
9. Report

Figure 1.2  Slope Stability Flowchart in Jointed Hard Rock for Surface Mining
rock slopes are definition of the problem, site investigation, and stability analysis in order to identify the modes of failure. The final stage is stabilisation of slopes and remedial works. It may be noted that the scope of this thesis extends up to identifying the mode of failures and suggestions for stabilising the slopes. Risk analyses, slope stabilisation and slope monitoring is beyond the scope of this thesis.

1.9 RESEARCH OBJECTIVE

Methods of rock mass characterisation are the basis of empirical slope stability design in surface mining. This research is concerned with the identification of important geometrical and mechanical parameters of rock slopes and the analysis of field data to assess mode of slope failure. It is also concerned with carrying out laboratory tests on intact rock samples and discontinuities, so as to obtain empirical co-relationship between the intact rock properties and that of rock mass. The main objective of the research is stability assessment in three different types of hard rock quarries in order to identify the potential modes of failure in these quarries. It is concerned with the statistical analysis of discontinuity data in order to identify the most important factor affecting hard rock slope stability and to study the relationship and interactions between discontinuity parameters contributing on the stability of hard rock slopes in surface mines.

1.10 RESEARCH SCHEDULE

The main purpose of this investigation is to study the application of rock mass characterisation in an approach to slope stability problems in open pit mines, together with a consideration of stability analysis methods.

This research consisted of five distinct phases, shown in Table 1.5. In the first phase of this research a literature survey of the fundamentals of rock slope stability problems, rock mass classification and consideration of factors affecting the slope stability was carried out.
Table 1.5 Schedule of the research.

<table>
<thead>
<tr>
<th>Description</th>
<th>2nd session</th>
<th>1st session</th>
<th>2nd session</th>
<th>1st session</th>
<th>2nd session</th>
<th>1st session</th>
<th>2nd session</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Literature review of rock slope stability problems</td>
<td></td>
<td>92</td>
<td>93</td>
<td>93</td>
<td>94</td>
<td>94</td>
<td>95</td>
</tr>
<tr>
<td>2 Site investigations, sampling and joint surveys at different type of quarries</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 Preparation of samples and laboratory testing of different hard rock specimens</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 Stability analysis of slope faces in order to identify modes of failure and possible remedial measure</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 Preparation of the thesis, conclusion and recommendations</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The second phase deals with the site investigations, sampling and joint surveys in three different type of quarries.

The third phase was involved with the preparation of samples and laboratory testing of different hard rock samples for the acquisition of input data used in the stability analysis of slope faces of the quarry.

The fourth phase was data analysis for stability assessment of slope faces in different type of hard rocks (porphyry, limestone and basalt quarries) in NSW.

The fifth and final phase was involved with the preparation of the thesis, to introduce general conclusions and recommendations for future research.
1.11 THESIS SCOPE

This thesis consists of eight chapters. The first chapter presents a consideration of rock slope stability problems relevant to quarries and open pit mines in Australia.

The most important factors affecting the instability of slopes are discussed at this stage. Figure 1.3 illustrates the structure of the eight chapters of this thesis.

Chapter 2 consists of a literature review of rock mass characteristics and their effects on the slope stability of rock slopes. Furthermore, in this chapter, aspects of rock mass classification and the effect of physical characteristics of discontinuities on rock slope stability is presented.

The third chapter of this thesis deals with methods of sampling, the preparation of samples, data acquisition and the estimation of rock mass strength from laboratory testing. The geological conditions of the area and the effect of intact rock properties on slope stability are presented.

The fourth chapter of this research consists of the shear strength testing of rock and rock mass. The shear strength of jointed rock samples is considered. For this purpose, block samples consisting of natural joints were tested in the departmental laboratory.

The fifth chapter deals with the assessment of slope stability in the surface mining of a Porphyry rock mass (Mugga II Quarry). The main aim of this chapter is to describe the means of obtaining the required parameters relating to the rock mass and structural features to aid the slope stability assessment for the surface mining.

Chapter six presents a geological investigation and appraisal of slope stability in connection with the application of rock mass characterisation to slope stability evaluation in a Limestone quarry (Marulan Quarry). In this chapter, data from discontinuity mapping, engineering properties of intact rock samples and other laboratory tests have been used for slope development analysis.

Chapter seven deals with the application of modified rock mass classification systems on the stability assessment of slope faces in a basalt quarry at Dunmore.
| CHAPTER ONE | Rock Slope Stability Problems in Hard Rock Quarries |
| CHAPTER TWO | Rock Mass Characteristics and Their Effects on Slope Stability |
| CHAPTER THREE | Data Acquisition and Estimation of Rock Mass Strength from Laboratory Testing |
| CHAPTER FOUR | The Shear Strength of Testing Rock and Rock Joints |
| CHAPTER FIVE | Stability Assessment of Surface Mining Slope in Porphyry Rock Mass |
| CHAPTER SIX | Geological Investigation and Appraisal of Slope Stability of a Limestone Quarry |
| CHAPTER SEVEN | Application of Modified Rock Mass Classification Systems on Stability Assessment of Slope Faces of Basalt Quarry |
| CHAPTER EIGHT | General Conclusions and Recommendations for Future Works |

Figure 1.3 Pattern of Thesis Structure
In this chapter, the physical properties of intact rock samples and data from discontinuities mapping were employed for the purpose of stability assessment of slope surfaces of the quarry.

Chapter eight presents the general conclusions of this research and recommendations for further research.
Chapter 2
Effects of Rock Mass Characteristics on Slope Stability
CHAPTER TWO

EFFECTS OF ROCK MASS CHARACTERISTICS ON SLOPE STABILITY

2.1 INTRODUCTION

In contrast to the foundation engineer, who is generally concerned with a specific site of limited extend, the slope engineer may be involved in designing many kilometres of highway cuttings or the overall slopes of an open pit mine. Since neither the time scale nor the economics of such a project allows a detailed investigation of each slope, it is essential that the slope engineer should work to a system which allows him to eliminate stable slopes at a very early stage of his investigations and to concentrate his attention onto those slopes which are critical (Hoek, 1974). There are limits to the applicability of rock mechanics to real rock engineering problems. These limits are set by the nature and extent of engineered rock structures, the variability of rock masses, and a general inability to achieve sufficient rock mass data for complete and rigorous analysis (Schmidt, 1992).

The presence of geological discontinuities is of paramount importance for the stability of rock slopes. Therefore, adequate data concerning the properties of discontinuities and the rock mass are required for an analysis leading to a numerical factor of safety.

This chapter consists of a literature review concerned with the discontinuities characteristics of the rock masses and factors affecting the stability of rock slopes in surface mining operations. In addition the procedures for data acquisition relating to
CHAPTER TWO

Effects of Rock Mass Characteristics on Slope Stability

rock masses and consideration of geotechnical parameters for the intact rock as well as in assessing the stability of rock slopes are presented.

In this chapter a number of methods for representing and analysing rock mass discontinuities data regarding to rock slope stability analysis purposes have been reviewed. Also the application of a computer method to the lower hemispherical projection technique has been examined. This is a quantitative technique employed in surface mining operations for analysis of the characteristics of potential modes of failure within a rock mass.

The main aim of this chapter can be summarised as follows:

- To evaluate geomechanical properties of rock mass discontinuities and their effects on instability of rock slopes.
- Review of some of the rock mass classifications and their application in rock slope stability analysis.
- To establish Rating Adjustment Parameters (RAP) for the rock mass classification systems and modification of rock mass rating (RMR) and rock mass quality (Q) system in order to assess and optimise current slope stability practice in surface mining operations.


2.2 FACTORS AFFECTING THE ASSESSMENT OF ROCK MASSES FOR ENGINEERING PURPOSES

The principle of a rock quality index is that a numerical assessment is made of the parameters controlling rock mass behaviour resulting in an index which is a function of the rock material and the discontinuities and can be defined as some relationship between these parameters (Houghton, 1976). Before a review of rock mass classification systems, a consideration of the factors affecting the rock masse behaviour
CHAPTER TWO  
Effects of Rock Mass Characteristics on Slope Stability

for engineering purposes is essential. It is obvious that different rock engineering projects will be influenced by different rock mass properties. In this regard the most important aspects will be selected using the skill and experience of the geologist and the geotechnical engineer directly involved. For engineering purposes and in any assessment of rock masses, three main aspects of the geological environment are as follows:

- Geological structures such as various types of discontinuities (faults, joints, bedding planes, folds, shear zones)
- Distribution of regional stresses
- Hydrological conditions

2.2.1 Geological Factors

The effect of geological structures on the engineering behaviour of rock masses can be divided into two categories:

(i) Influence of properties of discontinuities on rock mass behaviour
(ii) Importance of the regional structural geology on the engineering behaviour of rock masses. This factor is very important in engineering projects.

An analysis of the influence of regional structure on rock quality is, by necessity, of a very generalised nature, of course more rigorous analyses must be interpreted within the context of local geology. Standard techniques of geological mapping are readily available to help in elucidating geological structure and should form a part of any rock mechanics investigation program, (Houghton, 1976).

2.2.2 Distribution of Regional Stresses

In the assessment of rock masses for engineering purposes both the distribution and magnitude of regional stresses are important. The regional stresses have significant effects in the case of underground excavations. In each engineering project it is essential to make local measurements or assessments of the stresses and to evaluate the effect of the engineering structures on the existing distribution.
2.2.3 Hydrological Conditions

Although ground water plays a significant role in the behaviours of rock masses, and is one of the most important factors controlling rock slope stability, the existence of ground water is often taken for granted. Consideration of the effect of ground water on rock slope stability is essential for the analysis or design of a rock slope. Figure 2.1 shows in a simple way, the different forces which influence the instability of a rock block. However, the real ground water effects are often more complex, this figure shows the pressure of water in a tension crack.

![Diagram of forces on a rock block](image)

**Figure 2.1 Different forces on a detached block on rock slope.**

In any rock quality assessment concerning an excavation the following points should be incorporated:

- Rate of inflow of water into an excavations
- Duration and total anticipated volume of inflow
• Rock mass permeability and the effects of continuity of flow on permeability
• Ground water properties, temperature and chemistry
• Seepage forces and pressure developed.

2.3 PHYSICAL CHARACTERISTICS OF DISCONTINUITIES USED IN ROCK MASS CLASSIFICATIONS

2.3.1 Discontinuities

Generally discontinuities are geological structures which have been developed during the past geological periods. Discontinuity is the collective term for most types of joints, weak bedding planes, weak schistosity planes, weak mass zones and faults (ISRM, 1981).

Instability can occur in a rock slope as a result of slip or sliding failure along the major discontinuities in the rock mass. Discontinuities are important because they can be potential failure planes and also because they control the ground water condition of the rock mass. The importance of discontinuity characteristics to slope instability have been discussed by many authors, including Patton and Deere (1970), Goodman (1976), ISRM (1981), Khanlari and Jalaly, (1994); Bell (1983), Farmer (1983), Whittaker et al, (1992), Khanlari, (1989); Franklin and Dusseault, (1989). Figure 2.2 shows a typical joint survey data sheet which is prepared for evaluation of discontinuity characteristics concerned with stability assessment of a rock slopes. In the following sections, major effective factors on physical and mechanical properties of discontinuities are considered.

2.3.1.1 Different types of discontinuities

There are many different types of discontinuities and each type can influence the potential instability of a rock slope. Recognition of characteristics of different types of discontinuities and their geotechnical properties is necessary for stability evaluation. However, the most important types of discontinuities are as follows:

• Joints
• Bedding planes
<table>
<thead>
<tr>
<th>No</th>
<th>Dip</th>
<th>D. Direction</th>
<th>Joint Types</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2.2. Typical joint survey data sheet for evaluation of discontinuity characteristics.
• Shear zones
• Schistosity planes
• Contact zones
• Dykes

2.3.1.2 Physical characteristics of discontinuities

As mentioned above, in the field of instability of rock slopes, discontinuities have a principle role and often a slide is most likely to occur along the major discontinuity planes. According to ISRM, (1981) the most important physical characteristics of discontinuities are as follows:

(i) Number of joint sets

The number of joint sets is one of the most important factors influencing the block size. A joint set consists of individual joints with similar physical and mechanical characteristics that occur in a nearly parallel array. The jointing in a region can usually be subdivided into two or more such sets, which together constitute the jointing system. Often each set is characteristically different from the others because of differences in geological origin and history (Franklin and Dusseault, 1989).

Both the mechanical behaviour and the appearance of a rock mass will be dominated by the number of sets of discontinuities that intersect one another. The number of sets of discontinuities may be a dominant feature of rock slope stability, though traditionally the orientation of discontinuities relative to the face is considered of primary importance (ISRM, 1981). Joint sets are distinct and easily defined when the clusters of poles are tight. This is usually the case in sedimentary rocks little affected by folding or faulting. Bedding joints, for example are often near horizontal (Franklin and Dusseault, 1989).

The number of joint sets should be recorded during the field investigation, and systematic joint sets should be differentiated from non-systematic joints. The position of set number in a rock mass is shown in Figure 2.3. The number of joint sets occurring locally (for example along the length of a tunnel) is described by ISRM, (1981). Table 2.1 shows division of the number of joint sets.
The number of joint sets occurring locally (for example along the length of a tunnel) is described by ISRM, (1981). Table 2.1 shows division of the number of joint sets.

Table 2.1 Description of number of joint sets (After ISRM, 1981)

<table>
<thead>
<tr>
<th>NO:</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Massive, occasional random joints</td>
</tr>
<tr>
<td>II</td>
<td>One joint set</td>
</tr>
<tr>
<td>III</td>
<td>One joint set plus random</td>
</tr>
<tr>
<td>IV</td>
<td>Two joint sets</td>
</tr>
<tr>
<td>V</td>
<td>Two joint sets plus random</td>
</tr>
<tr>
<td>VI</td>
<td>Three joint sets</td>
</tr>
<tr>
<td>VII</td>
<td>Three joint sets plus random</td>
</tr>
<tr>
<td>VIII</td>
<td>Four or more joint sets</td>
</tr>
<tr>
<td>IX</td>
<td>Crushed rock, earth like.</td>
</tr>
</tbody>
</table>
(ii) Orientation of discontinuities

The orientation of a discontinuity in space is described by the dip of the line of steepest inclination measured from horizontal, and by the dip direction measured clock-wise from true north (ISRM, 1981). Example: dip direction / dip (035/60) In the well-known dip-strike notation used by geologists, strike is the direction of a horizontal line in the plane of the joint, and dip is the vertical angle measured downward from the horizontal to the fall line of greatest dip (Franklin and Dusseault, 1989).

Dip magnitude $\beta$ is the same as defined above, expressed as a two-digit number. Dip direction $\alpha$ is always expressed as a three-digit azimuth between 0 and 360° measured clock-wise from north to the horizontal projection of the fall line. Figure 2.4 shows dip direction $\alpha$ and dip magnitude $\beta$. The combined dip and dip direction thus appearing in the rock description in the form "18° at 025°".

Figure 2.4 Illustration of dip direction $\alpha$ and dip magnitude $\beta$

The orientation of discontinuities plays an important role in the rock slope instability and the methods of measurement of discontinuity orientation have been discussed by the following authors, Goodman, (1976), Hoek & Brown (1980), Hoek & Bray (1981), ISRM. (1981), Franklin, (1989), Young, (1993), He, (1994), Maerz et al (1990), Villaescusa et al, (1992). Figure 2.5 shows the dip, dip direction and strike of three differently orientated planes (After ISRM, 1981.).
Figure 2.5 Diagram indicating the strike, dip and dip direction of three differently oriented planes (After ISRM, 1981).
(iii) Aperture

Aperture is the perpendicular distance separating the adjacent rock walls of an open discontinuity, in which the intervening space is air or water filled (ISRM, 1981). In other words, the aperture of a joint (also known as its openness or separation) is the mean distance separating the two intact joint walls. Note that aperture includes the thickness of any filling that may be present (Franklin and Dusseault, 1989).

Aperture is one of the most important physical parameters of discontinuities, and has an influence on the permeability and shear strength of the rock mass. The effect of this parameter on the permeability of the rock mass has been studied by many authors, as discussed in Hoek & Bray (1980).

Influenced by the type of infilling material (clay minerals) the potential for the sliding of rock slopes may be increased. In contrast some materials such as quartz and hard minerals help to decrease the potential for sliding. Usually variation of the width of aperture is between just a few micrometers to probably a few centimetres. According to the ISRM, (1981) aperture can be measured using the following equipment:

- Measuring tape of at least 3 m length, calibration in mm.
- Feeler gauge for estimating the width of fine apertures.
- White spray paint
- Equipment for washing the rock exposure.

Typical aperture dimensions is introduced in Table 2.2

Table 2.2 Description of divisions of aperture

<table>
<thead>
<tr>
<th>Aperture</th>
<th>Description</th>
<th>Feature</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 0.1 mm</td>
<td>Very tight</td>
<td>&quot;Closed&quot; feature</td>
</tr>
<tr>
<td>0.1 - 0.25 mm</td>
<td>Tight</td>
<td>&quot;Gapped&quot; feature</td>
</tr>
<tr>
<td>0.25 - 0.5 mm</td>
<td>Partly open</td>
<td></td>
</tr>
<tr>
<td>0.5 - 2.5 mm</td>
<td>Open</td>
<td></td>
</tr>
<tr>
<td>2.5 - 10 mm</td>
<td>Moderately wide</td>
<td></td>
</tr>
<tr>
<td>&gt; 10 mm</td>
<td>Wide</td>
<td>&quot;Open&quot; feature</td>
</tr>
<tr>
<td>1 - 10 cm</td>
<td>Very wide</td>
<td></td>
</tr>
<tr>
<td>10 - 100 cm</td>
<td>Extremely wide</td>
<td></td>
</tr>
<tr>
<td>&gt; 1 m</td>
<td>Cavernous</td>
<td></td>
</tr>
</tbody>
</table>
(iv) Filling Materials

Filling material can be defined as any material separating the adjacent rock walls of discontinuities such as clay, calcite, quartz, etc. The perpendicular distance between the adjacent rock walls is termed the width of the discontinuity, as opposed to the aperture of a gapped or open feature (ISRM, 1981).

Filling materials are very different and vary greatly in their mechanical behaviour, from very hard and strong to very soft and weak. Filling material plays a vital role in the stability of rock slopes. The thickness of filling material also has a great influence on the shear strength of the rock mass. When the thickness of filling material is more than the amplitude of roughness of a discontinuity, the strength of the filling material dominates the strength of the discontinuities. Thicknesses of filling materials can be estimated visually and expressed in terms such as "thinner than 1 mm". Intermediate thicknesses should be expressed as a range and a typical value, for example "(1-10) 3 mm", whereas substantial thicknesses generally require that the overall thickness value be accompanied by a sketch showing the types and layers of materials (Franklin and Dusseault, 1989).

(v) Roughness

In general terms, the roughness of discontinuity walls can be defined by a "waviness" (large scale) and by an "unwaviness" (small scale). In practice waviness affects the initial direction of shear displacement relative to the mean discontinuity plane, while unwaviness affects the shear strength that would normally be sampled in a laboratory or medium scale in situ direct shear test (ISRM, 1981).

To observe and measure roughness, a shadow can be cast on a rough rock joint facet by a straightedge held against the surface in bright sunlight, using photo-analysis, the edge of the shadow can be digitized and any required roughness parameter calculated (Franklin, Maerz and Bennett, 1987).

The quantitative description of joint roughness and its effect on the shear strength behaviour and dilation of joints has been expressed by Barton (1973); ISRM, (1981); Reaves (1985); Maerz et al, (1990); Jermy, (1994). Also for characterising roughness Reaves has introduced following equation.
\[ Z_2 = \frac{1}{L} \int_{x=0}^{x=L} \left( \frac{dy}{dx} \right)^2 \]  

(2.1)

Where \( Y \) is the amplitude at a particular point on the roughness profile about the centre line, \( X \) is the tangential distance along the profile, and \( L \) is the total length of the profile.

Typical roughness profiles of the nine classes suggested by ISRM, (1981) are illustrated in Figure 2.6.

(vi) Spacing

Spacing can be defined as the average distance between adjacent joints in a set, measured normal to the joint plane. The spacing of adjacent discontinuities largely controls the size of individual blocks of intact rock.

As in the case of orientation, the importance of spacing increases when other conditions for deformation are present, i.e. low shear strength and a sufficient number of discontinuities or joint sets for slip to occur. The spacing of individual discontinuities and associated sets has a strong influence on the rock mass permeability and seepage characteristics (ISRM, 1981).

A measuring tape of at least 3 m length, calibrated in mm division and a compass is necessary for the determination of joint spacing. The measuring tape should be held along the exposure such that the surface face of the discontinuity set being measured is approximately perpendicular to the tape. According to the ISRM, (1981) the most common spacing can be calculated from the following equation:

\[ S = d_m \sin \alpha \]  

(2.2)

Where \( d_m \) is the most common distance measured and \( \alpha \) is the smallest angle between the measuring tape and the observed joint set. Figure 2.7 shows joint spacing measurement in a rock mass.
Figure 2.6 Illustration of typical roughness profiles and suggested nomenclature (After ISRM, 1981).
Fig. 2.7 Joint spacing measurement of rock mass.
The international society of rock mechanics (ISRM, 1981) has introduced the following classification for the description of spacing shown in Table 2.3

Table 2.3 Description of joint spacing (After ISRM, 1981)

<table>
<thead>
<tr>
<th>Description</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely close spacing</td>
<td>&lt; 20 mm</td>
</tr>
<tr>
<td>Very close spacing</td>
<td>20 - 60 mm</td>
</tr>
<tr>
<td>Close spacing</td>
<td>60 - 200 mm</td>
</tr>
<tr>
<td>Moderate spacing</td>
<td>200 - 600 mm</td>
</tr>
<tr>
<td>Wide spacing</td>
<td>600 - 2000 mm</td>
</tr>
<tr>
<td>Very wide spacing</td>
<td>2000 - 6000 mm</td>
</tr>
<tr>
<td>Extremely wide spacing</td>
<td>&gt; 6000 mm</td>
</tr>
</tbody>
</table>

(vii) Persistence

Persistence or continuity can be defined as the percentage ratio of joint fragment to total area measured in the plane of the joint. In the case of rock slopes, persistence plays a significant role. The degree of persistence of those discontinuities that are unfavourably orientated for stability of the rock slope is, therefore, very important and even vital.

For measuring persistence a tape of at least 10 m length is necessary and individual rock exposure or recognised domains should first be described according to the relative persistence of the different discontinuity sets present (After ISRM, 1981).

In sliding stability calculations, small intact rock bridges have a great affect on total shear strength of a discontinuity. Therefore a design engineer needs to know what percentage these represent of the total surface of sliding. Table 2.4 shows the description of persistence and also in Figure 2.8 the persistence of the various sets of discontinuities is illustrated.
Figure 2.8 Persistence of the Various Sets of Discontinuities (After ISRM, 1981).

Table 2.4 Description of Persistence (After ISRM, 1981).

<table>
<thead>
<tr>
<th>Description</th>
<th>Persistence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very low persistence</td>
<td>&lt; 1 m</td>
</tr>
<tr>
<td>Low persistence</td>
<td>1 - 3 mm</td>
</tr>
<tr>
<td>Medium persistence</td>
<td>3 - 10 mm</td>
</tr>
<tr>
<td>High persistence</td>
<td>10 - 20 mm</td>
</tr>
<tr>
<td>Very high persistence</td>
<td>&gt; 20 m</td>
</tr>
</tbody>
</table>
(viii) **Block size**

In general, each rock mass has a characteristic block shape which depends on the number of joint sets and their relative orientations and spacing. Block size is an important factor in rock masses and the shape of the blocks depends on the number of joint sets, for example cubic blocks are produced by three dimensional and equally spaced joint sets and slabby blocks produced from a single closely spaced joint set and two at a wider spacing. Figure 2.9 shows different types of block shapes.

The number and orientation of joint sets determines the shape of the resulting block, which can take the approximate form of cubes, rhombohedrons, tetrahedrons, and sheets. However regular geometric shapes are the exception rather than the rule since the joints in any one set are seldom consistently parallel. Jointing in sedimentary rocks usually produces the most regular block shapes. Rock quarrying and blasting efficiency is likely to be largely a function of the natural in-situ block size (ISRM, 1981).

For the measuring of the block size a tape of at least 3 m length, calibrated in mm divisions is necessary and block size can be described by the following ways:

- By means of the average dimension of typical blocks ($I_b = \text{Block size index}$)
- By the total number of joints intersecting a unit volume of the rock mass ($J_r = \text{Volumetric joint count}$).

The volumetric joint count can be defined as the sum of the number of joints per metre in each joint set present, also random discontinuities can be included. For the determination of $J_r$, the number of joints of each set should be counted along the relevant joint set perpendicular and also a sampling length of 5 or 10 m is suggested by ISRM, 1981. For example for three joint sets and a random discontinuity counted along 5 or 10 m perpendicular spacing lines might appear as follow:

$$J_r = \frac{4}{10} + \frac{18}{10} + \frac{4}{5} + \frac{3}{10} \quad (2.3)$$

$$J_r = 0.4 + 1.8 + 0.8 + 0.3 = 3.3/m^3 \quad \text{(Medium size block)}$$

The objective of the block size index is to represent the average dimensions of typical rock blocks. The average value of individual model spacings ($S_1, S_2, S_3$, see spacing) may not introduce a realistic value of $I_b$ if there are more than three sets.
The fourth set, if widely spaced, will artificially increase $I_b$, but may have little effects on real block sizes as observed in the field. But in the case of sedimentary rocks, two mutually perpendicular joint sets of cross joints plus bedding constitute an extremely common cubic, so $I_b$ can be measured by following equilibrium:

$$I_b = \frac{S_1 + S_2 + S_3}{3} \quad (2.4)$$

Descriptions of block size are given in Table 2.5.
Table 2.5 Description of block size (After ISRM, 1981)

<table>
<thead>
<tr>
<th>Description</th>
<th>J_r (Joint / m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very large blocks</td>
<td>&lt;1.0</td>
</tr>
<tr>
<td>Large blocks</td>
<td>1 - 3</td>
</tr>
<tr>
<td>Medium-sized blocks</td>
<td>3 - 10</td>
</tr>
<tr>
<td>Small blocks</td>
<td>10 - 30</td>
</tr>
<tr>
<td>Very small blocks</td>
<td>&gt; 30</td>
</tr>
</tbody>
</table>

Note that block size has great influence on the instability of rock slope, particularly in the sedimentary rocks (bedded rock) and when the beddings are not horizontal.

(ix) Wall strength

Among the mechanical properties of rock masses the shear strength of discontinuities is very important and is affected by weathering and alteration. The compressive strength of the rock comprising the walls of a discontinuity is a very important component of shear strength and deformability, especially if the walls are in direct rock to rock contact as in the case of unfilled joints (ISRM, 1981). The strength of non altered wall surfaces is similar to the strength of the intact rock, this parameter can be determined directly by measuring uniaxial compressive strength or can be obtained indirectly and more quickly from the point-load strength of core samples.

When joint walls are weathered or altered, the weathering and alteration generally affects the walls of discontinuities more than the interior of rock blocks. On the other hand when joint walls are coated thickly with materials which are products of weathering or alteration, the filling materials can be sampled and tested in the laboratory, in this case shear strengths of filling materials may be determined.

For measurement of the joint wall compressive strength (JCS), Schmidt's hammer (L type) with conversion table and graph can be used when utilising the wall roughness coefficient (JRC) described under roughness. Weakened wall rocks can be tested in the laboratory or the field using a Schmidt hammer, a simple and portable testing device. JCS is then estimated as follows:
\[
\log_{10}(JCS) = 0.00088\rho R + 1.01
\]  \hspace{1cm} (2.5)

Where \( JCS \) is in MPa, \( \rho \) is the dry density in kN per cubic metre, and \( R \) is the Schmidt rebound number of the joint surface (Franklin and Dusseault, 1989).

(x) Seepage

Water seepage through rock masses results mainly from water conducting through discontinuities. The prediction of ground water levels, likely seepage paths, and approximate water pressures may often give advance warning of stability or construction difficulties (ISRM, 1981).

Water pressure plays a principle role in the instability of a rock mass particularly when filling materials are clay minerals. It should be noted that in the field investigation of rock masses, amounts and locations of observed seepages must be described.

In the case of rock slopes, the preliminary design estimates will be based on assumed values of effective normal stress. If, as a result of field observations one has to conclude that pessimistic assumptions of water pressure are justified (i.e. a tension crack full of water with zero exit pressure at the toe of the unfavourable discontinuity) then this will clearly have the greatest consequences for design (ISRM, 1981). In the definition of joint seepage condition, terms such as dry, moist, slight, or moderate seepage, dripping and flowing can be used.

(xi) Weathering

Weathering is the process of weakening and/or disintegration of rocks under the influence of the atmosphere and hydrosphere (Fookes, et al; 1971). In other words weathering is a process by which rock is broken down and decomposed due to the combined effect of environmental factors such as, temperature, air, water and chemical activities. Table 2.6 shows a description of rock weathering classification.

The gradational change of bed-rock to soil is due to weathering (Carroll, (1977); Hunt, (1972); Rahn, (1986); and Franklin, et al (1989)).
Weathering processes can be classified as physical, chemical and biological. When the weathering takes place along the walls of discontinuities, it can reduce the wall strength and also RQD of rock mass.

2.4 SCALE EFFECT OF THE ROCK STRENGTH AND DEFORMABILITY

There are different methods for studying the significance of discontinuities on the behaviour and deformability of the rock masses. Hook and Brown (1980) illustrated the transition from intact rock to a heavily jointed rock mass with increasing sample size in a hypothetical rock mass surrounding an underground excavation. It should be noted that, this problem is not only for underground excavations but also includes, rock cuts and open pit mining operations.

The role of scale effect on the stability of a rock slopes in open pit mines is presented in Figure 2.10. Although, most stability analyses assume simple gravitational body loading to calculate the stress on a failure surface. It is recognised that the actual stress magnitude and orientation of discontinuities are affected by the in-situ stress field, the geometry of the pit, and the variation in material properties. There is a stress concentration at the toe of a slope that is a result of the deflection of stresses around the toe. A high horizontal stress produces a greater stress than simple gravity loading.

The effects of in-situ stress in surface mines arise from abnormally high stresses. As the rock is excavated, the weight of the rock is removed, and the remaining rock in the walls and the floor rebounds and deforms into the excavation, the result is loosening of the rock and in competent strata, uplift and detachment of the immediate floor. The effects of in-situ stress on the rock mass has also been noted as an increase in water-well yield near surface mines that is attributable to stress relief in the rock mass and the consequent opening of fractures in aquifers, (Call, 1992).
Figure 2.10 Illustration of the effect of major and minor joint systems on the stability of rock slope faces in an open pit mine.
2.5 DIFFERENT METHODS FOR DATA COLLECTION OF DISCONTINUITIES

There are many different methods for evaluation of rock mass discontinuities. In rock engineering, particularly in the case of rock slope stability, a definition of the engineering properties of the discontinuities is essential, because the instability of rock slopes is highly dependent on the properties of discontinuity planes. The stability analysis of rock slopes involves different methods and techniques in relation to joint data collection. The most important methods are discussed in later sections.

2.5.1 Scan-line Method

The scan line method is recommended by ISRM (1981). This method consists of stretching a measuring tape or a line along the slope face and all of the discontinuities which intersect the tape or line must be measured. Therefore, when the scan line method is employed properties of discontinuities such as dip and dip direction, roughness, spacing, infilling, persistence, aperture and joint compressive strength must be measured of quantitatively and qualitatively.

2.5.2 Area Mapping Method

Area mapping method is another method for data collection relating to properties of discontinuities and this method involves the observation of all existing joints on the slope faces in an open pit mine. This method can be employed when the scan line is not suitable for collecting data relating to discontinuities. Mapping of visible structural features on outcrops or excavated faces is a slow and tedious process but unfortunately there are few alternatives to the traditional techniques used by the geologist. The most important and currently used tool for mapping is the geological compass. The principle types include the Clar type and the Brunton type commonly used for joint surveys. In addition, Pothoanalysis method was proposed by Andtsoutrelis et al, (1990) in ordr to stydu the rock mass discontinuity systems.
2.6 MECHANICAL PROPERTIES OF INTACT ROCK

Intact rock can be defined as the rock material which is removed from its environment and which is free from discontinuities. In rock engineering the role of strength of intact rock is very important particularly if normal stresses are high and surface roughness is significant. During the last three decades intact rock properties have been studied by many different authors (Deere et al, (1966); Attewell and Farmer, (1976); Goodman, (1989); Hoek and Brown, (1980); Hoek and Bray, (1981); Farmer, (1983); Bell, (1980, 1983); Brady and Brown, (1985) and Franklin, and Dusseault (1989). In most cases of slope instability, it has been observed that the instability of the slopes rarely occurs in the intact rock material and failures are usually due to the rupture of rocks along weak zones of the rock mass or the discontinuities. The most important rock material properties which can be collected from observation of the rock mass are as follows:

2.6.1 Rock Types and Lithology

Classical rock names used without qualification can be misleading in an engineering context (Franklin and Dusseault, 1989). Generally for an engineering considerations, rock description and geological name of rock should be reported according to ISRM commission on classification (engineering names). Name and type of rock can probably represent the engineering properties of the rock. For example shale implies fissility compared to quartzite which suggests a massive crystalline rock. It is obvious that the mechanical properties of rock are dependant upon the mechanical properties of the constituent minerals. Other properties of intact rock include the following:

- Colour

This parameter can be used not only for identification of rock type but also as an indication of the chemical composition of the rock, for example white coloured rocks are acidic in comparison with black coloured rocks which are generally basic.

- Moisture content

This is a significant factor influencing the strength of rock and also its weathering properties.
• Hardness

The concept of hardness is applied loosely to rock as a synonym for strength (e.g. soft rock and hard rock) but hardness is more strictly defined as a property of metals and of the rock-forming minerals (Franklin and Dusseault, 1989). This parameter is usually used as a direct indication of strength, although some believe this is not good practice (Houghton, 1976).

• Weathering

This parameter has great influence on the behaviour of rocks. During the last three decades different classifications of weathering have been introduced by different authors.

• Strength of intact rock

Uniaxial compressive strength is one of the most important engineering factors of rock materials and can be measured directly by laboratory tests or indirectly estimated from the results of point load testing.

• Porosity and Density

From an engineering point of view these two parameters are very important because they effect some of the physical properties such as strength, deformability and hydraulic storage. Rock material can be divided into three categories based on porosity, these being dense, porous and very porous. Micro fissured rocks are usually weaker than porous ones without micro fissures but with same total porosity (Kelsall et al., 1986).

2.6.2 Strength

The strength of intact rock can be defined as the maximum normal force per unit area which a rock sample can bear until yield point. This characteristic is recognised as the compressive strength and has been made the basis for classification of intact rocks. For many years the uniaxial or unconfined test was the main quantitative method for characterising the strength of rock materials (Hawkes and Mellor, 1970). For example Mount Isa mines have performed more than 20,000 uniaxial compressive strength tests during a 30 year period (Franklin and Dusseault, 1989). Based on this characteristic,
intact rocks can be categorised from strong to weak. For assessing the intact rock strength several index tests have been introduced, these are as follows:

- Point load index
- Brazilian test
- Schmidt hammer test
- Shore scleroscope
- NCB cone indenter

It should be noted that the strength of intact rock is dependant upon different factors which are as follows:

- Lithology (Mineral composition)
- Degree of weathering
- Density
- Porosity
- Water content
- Durability, Plasticity and Swelling potential

2.6.3 Anisotropy

Anisotropy of intact rock properties is considerable and, because of this factor, engineering behaviour of intact rock varies in different directions. Therefore, it is vital to test the rock samples which are considered anisotropic in different orientations. In this case, often the strength of the rock is stronger in one direction than other.

2.6.4 Weathering

Weathering of rocks occurs under the influence of the hydrosphere and the atmosphere as a result of their decomposition in environments which differ from those in which they were formed. Although weathering is a slow process, the main influence of weathering is a decrease of strength, density, volumetric stability, and also an increase of deformability
and porosity over long times. Weathering processes can be classified into the physical, chemical and biological. When the weathering occurs along the walls of the discontinuities, the wall strength and RQD of rock mass will be reduced.

In the discontinuities, fractures are stained or decomposed and may contain a thin filling of weathered and altered materials. Stains and discolouration of rock may extend into the rock mass from the discontinuities planes to a distance of up to 20% of the fracture spacing, i.e. less than 40% of the core is discoloured. (Core logging committee of South Africa, 1976). Weathering has great influence on the instability of rock slopes, and in the intact rock. The rock quality designation (RQD) will be reduced as a result of the weathering process as well as rock strength.

In the process of weathering, generally both mechanical and chemical effects act together and it will depend upon the climate region which of these processes will dominate. Various weathering classifications have been introduced based on the extent of penetration from joints and the degree of decomposition of the intact rock materials (Brand and Phillipson, 1984). The slake durability test was developed by Franklin and Chandra, (1972) and can be used for assessing the effect of mechanical weathering on intact weak rocks. An engineering classification of weathering has been introduced by the Core Logging Committee of South Africa in 1976 and it has been shown in Table 2.6.

2.7 GRAPHICAL PRESENTATION OF DISCONTINUITY DATA ANALYSIS

2.7.1 Introduction

One of the most important aspects of rock slope stability analysis is the systematic collection and presentation of geological data so that it can easily be evaluated and incorporated into stability analyses of rock slopes.

Many characteristics of discontinuities such as orientation, strength of the discontinuities within the rock mass and geometry of slopes are effect the instability of jointed rock slopes. The engineering behaviour of rock materials is highly controlled by the predominant modes of failure and also by engineering properties of the intact rock and mechanical properties of discontinuities.
<table>
<thead>
<tr>
<th>Diagnostic Feature</th>
<th>Extent of Discolouration</th>
<th>Fracture Condition</th>
<th>Surface Characteristics</th>
<th>Original Texture</th>
<th>Grain Boundary Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unweathered</td>
<td>None</td>
<td>Closed or discoloured</td>
<td>Unchanged</td>
<td>Preserved</td>
<td>Tight</td>
</tr>
<tr>
<td>Slightly Weathered</td>
<td>&lt;20% of fracture spacing on both sides of fracture</td>
<td>Discoloured may contain thin filling</td>
<td>Partial discoloured</td>
<td>Preserved</td>
<td>Tight</td>
</tr>
<tr>
<td>Moderately Weathered</td>
<td>&gt;20% of fracture spacing on both sides of fracture</td>
<td>Discoloured may contain thick filling</td>
<td>Partial to complete discoloured not friable except poorly cement rocks</td>
<td>Preserved</td>
<td>Partial opening</td>
</tr>
<tr>
<td>Highly Weathered</td>
<td>Throughout</td>
<td>--------------------</td>
<td>Friable and possibly pitted</td>
<td>Mainly Preserved</td>
<td>Partial separation</td>
</tr>
<tr>
<td>Completely Weathered</td>
<td>Throughout</td>
<td>--------------------</td>
<td>Resembles a soil</td>
<td>Partly Preserved</td>
<td>Complete separation</td>
</tr>
</tbody>
</table>
2.7.2 Stereographic Projection

There are different methods for presentation and analysis of discontinuities data so, when there are a large number of discontinuity orientations, it is necessary to plot the orientation of each pole on a hemispherical projection. In order to present data orientation many types of spherical projection have been introduced by many authors; Turner and Weiss (1963), Phillips (1971), Hoek and Bray (1981), Priest (1984). The lower hemispherical projection method is a suitable graphical and analytical mode for presentation of failure types in open pit mine slopes.

Two main types of stereographic projections widely used for presentation of orientation data are as follows:

- Equal-angle projection or Wulf stereographic nets.
- Equal-area projection or Lambert-Schmidt nets.

Figure 2.11 shows the dip, dip direction and strike conventions used in relation with the lower reference hemisphere stereographic projection. In order to communicate the information given by the great circle and the position of the pole on the surface of the lower hemisphere, a two dimensional representation is obtained by projecting this information onto the horizontal or equatorial reference plane. The pole is the point at which the surface of the sphere is pierced by the radial line which is normal to the plane (Hoek and Bray, 1981). The method of projection is shown in Figure 2.12.
2.7.2.1 Equal - angle projection

The equal angle projection or stereographic projection was introduced by Wulf and this method offers certain advantages, particularly when employed for geometrical constructions. This method is recommended by many authors Terzaghi, (1965), Hoek and Brown, (1980), Hoek and Bray, (1981), Giani, (1992). The equal angle or stereographic projection is different from the equal area projection in many important respects. For presentation the structural geology, the traces of planes on the surface of a reference sphere are used to define the dips and dip directions of the planes. This system is used for representation of spherical shape of the earth in two dimensional. Figure 2.13 shows the stereographic projection. A polar stereographic net is shown in Figure 2.14.

2.7.2.2 Equal - area projection

This method was introduced by Lambert - Schmidt. In adapting this projection to structural geology the traces of planes on the surface of a reference sphere are used to define the dips and dip direction of the plane. Generally in engineering purposes, the
lower reference hemisphere is employed to present the data (Hoek & Bray 1981). Orientation of the plane can be defined by a great circle or by the pole of the plane which is the point that is normal to the plane. Figures 2.15 shows an equatorial equal area net. A Denness counting net and also a counting circle is shown in Figure 2.16.

Figure 2.12 Method of construction of an equal-area (a) and equal-angle projection (b) (After Hoek and Brown, 1980)

2.7.2.3 Evaluation of modes of failure

A conventional method for plotting field measurement of dip and dip direction is plot of poles, which can be plotted directly on a polar equal area stereonet which is calibrated in 2° intervals. In this projection when the amount of dip and dip direction of a joint plane is defined it is possible to draw directly the pole of the joint plane on a polar equal - area stereonet, for example if a joint plane dip direction and dip values are 080/ 30, the pole is located on the polar area stereonet by using the value of dip direction 080 given in italics and then separating the dip value of 30 from the centre of the polar equal area stereonet along the radial line. It is not necessary to rotate the tracing paper, centred over the stereonet net. As a result drawing the pole of the discontinuities can be done quickly. Figure 2.14 shows a polar equal - area stereonet in 2° intervals.
CHAPTER TWO  
Effects of Rock Mass Characteristics on Slope Stability

Figure 2.13  Meridional stereographic net or equal-angle stereonet (After Hoek and Brown, 1980)

Figure 2.14  Polar stereographic net or polar equal-area stereonet (After Hoek and Brown, 1980)
CHAPTER TWO  Effects of Rock Mass Characteristics on Slope Stability

Figure 2.15   Equatorial equal-area stereonet (After Hoek and Brown, 1980)

(a)  

(b)  

Figure 2.16  a) Denness counting net; b) counting circle (After Hoek and Bray, 1981)
2.8 INTEGRATION OF INTACT ROCK PROPERTIES INTO THE ROCK MASS CLASSIFICATION SYSTEMS

The history of rock mass classification refers to beginning in the writings of the ancient Greeks as did the first exploitation and mining works. The results of rock mass classification are applied to planning and programming of engineering works. It is widely established today that engineering classifications of rock masses are vital for assessing rock mass conditions. Therefore the classification of a rock mass is an important issue in the selection of the most suitable parameters for design in rock mass. For example uniaxial compressive strength is one of the most important of these factors and can be determined directly by laboratory tests or indirectly by the point load test.

During the time rock mass classification systems have been used in underground mining works such as tunnel support design and the assessing of rock slope stability in the open pit mines. Engineering classification attempts to assess the suitability of a rock mass for engineering purposes and consequently the selection of parameters for use in such classifications is of special importance.

2.8.1 Time Dependency

It has been observed that most rock slopes does not fail immediately after excavation, but the failure of the rock slopes generally occurs often during the time and usually after the excavation has been completed. This problem indicates that failure of rock slopes in open cast mining depends upon time elapsed after completion, since instability factors such as joint sets, weathering and alteration, tension cracks, pore water pressure takes place in the rock mass over a period of time and may lead to slope failure later.

Generally most of the geological processes will complicate over long time periods and deformation of intact material does not appear over a short time. Therefore the failure process of rock mass is very complex but, it is possible to have a good understanding of the behaviour of the rock mass by studying the rock mass characteristics.

2.8.2 Ground water

The effect of water pressure on the reduction of the strength of rock material can be seen since, when rock material becomes saturated, the strength of the rock can become
weakened. Generally, it is vital to consider the active water pressure at a failure surface when an analysis or design of a slope is undertaken. The water pressure in tension cracks increases linearly with depth and the total force \( V \), due to this water pressure acting on the rear face of the block, acts down the inclined plane. Because of the pore water pressure, the strength reduction in different types of rock varies.

Crystalline rocks such as granite and sandstone have a slight reduction of strength due to the presence of the water, whereas other types of rocks such as mudstones and marls shales show a large reduction in strength associated with the presence of water. Water pressure play a significant role in most slope failure, and it is probable that the failures on the slopes can be precipitated by an increase in pore water pressures in the loose rock and along joints in hard rocks (Brackley et al, 1989). Consequently consideration of the effect of ground water on rock slope stability is essential for an analysis or design of a rock slope. Figure 2.17 shows the effect of groundwater table on the instability of a soil or rock slope.

![Figure 2.17 Effect of groundwater table on soil and rock slope stability.](image)

where

\[ V = \text{water force} \]

\[ U = \text{uplift force} \]
W = weight of block

The condition of limiting equilibrium for this case of a block acted upon by water forces V and U in addition to its own weight W is defined by:

\[ W \sin \psi + V = CA + (W \cos \psi - U) \tan \phi \]  

(2.6)

From this equation it will be seen that the disturbing force tending to induce sliding down the plane is increased and the frictional force resisting sliding is decreased and hence both V and U result in decreases in stability (Hoek and Bray 1981).

2.8.3 Slope Geometry

The geometry of slopes has an essential role in the stability of the rock slopes, so that generally flat slopes are more stable than steeper slopes. The main aim of slope geometry adjustment is the control of damage or reducing the volume of the possible failure of the rock mass.

Changing of slope geometry can be done by benching and the final slope geometry can be determined based on consideration of the stability of the slope. The effect of slope geometry on the stability of a slope is illustrated in Figure 2.17

2.9 ENGINEERING CLASSIFICATION OF INTACT ROCK

An engineering classification of intact rock has been introduced by Deere and Miller, (1966) based on two parameters

- Uniaxial compressive strength
- Modulus Ratio

These factors are the most important physical and mechanical properties of intact rock which can be used for the solution of most engineering problems. This classification is a numerical value system and it has been widely used in engineering application. Modulus of elasticity, E is determined by uniaxial compressive strength tests from standard core.
In this classification rock materials have been classified into five categories according to compressive strength, and the higher values of compressive strength is more than 200 MPa and lower rate is less than 25 MPa. Also in this system rock materials are divided into three categories according to modulus ratio. In this classification the higher modulus ratio is greater than 500 and the lower ratio is less than 200. Modulus ratio is the ratio of compressive strength to elastic modulus. The Deere and Miller classification is shown in Table 2.7

Table 2.7 Engineering classification of intact rock (After Deere and Miller, 1966)

<table>
<thead>
<tr>
<th>Class</th>
<th>Description</th>
<th>U.C.S (MPa)</th>
<th>Modulus Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Very high strength</td>
<td>&gt; 200</td>
<td>&gt;500</td>
</tr>
<tr>
<td>B</td>
<td>High strength</td>
<td>100 -- 200</td>
<td>-</td>
</tr>
<tr>
<td>C</td>
<td>Medium strength</td>
<td>50 -- 100</td>
<td>200 - 500</td>
</tr>
<tr>
<td>D</td>
<td>Low strength</td>
<td>25 -- 50</td>
<td>-</td>
</tr>
<tr>
<td>E</td>
<td>Very low strength</td>
<td>&lt;25</td>
<td>&lt;200</td>
</tr>
</tbody>
</table>

2.10 ROCK MASS CLASSIFICATION REVIEW

Generally the engineering behaviour of intact rock depends on the properties of discontinuities, and predicted potential failures assist in defining the overall rock mass behaviour. The results of the rock mass classification have been used in the planning and programming rock engineering projects. During the last three decades many different rock mass classification systems have been developed. The classification of rock mass by selecting the greatest parameters of the rock mass is an important issue in rock engineering. For example, uniaxial compressive strength is one of the most important parameters of rocks which can be determined directly by laboratory tests or indirectly by the point-load test strength index (Franklin and Dusseault, 1989). During three last decades, rock mass classification systems have been studied by many authors (Van Der Vlis, (1970); Barton et al, (1976); Cockcroft(1976); Kendorski et al (1983); Nguyen et al, (1985); Turk et al, (1985); Al-Harthi, (1993).
2.10.1 Aspects of Rock Mass Classification

The importance of rock mass classification is to evaluate the major properties of the rock mass and to assign a numerical value to each parameter and to combine these values into a total classification rating value for the assessment of the rock mass. The main aim of rock mass classification systems in surface mining operations is to introduce a simple and effective classification for description of rock mass properties in relation to rock slope stability and other rock engineering purposes. Hoek, (1974) stated that the engineering appraisal of rock mass includes;

- a quantitative estimate of the response of the rock mass to change in either geometry or loading. This includes an assessment of possible failure modes.
- a quantitative measurement of parameters used in the numerical analysis of the behaviour of the slope or foundations

One of the principal applications of rock mass classification in surface mining is the consideration of the immediate and long term stability of structures which are excavated in rock masses. The objective natural characteristics of rock mass may be presented by three conditions; engineering geological rock group; rock mass structures and existent environment of rock mass.

Thus, the engineering properties of a rock mass can not be determined directly from laboratory tests on specimens of small volume which may not adequately reflect the in situ fracture pattern of the rock mass (Chinsman, 1977). Rock mass classification has been used on the surface mining by many researcher to assess stability of slope faces. There are many different rock mass classification systems which are used in the field of rock engineering. However, in this research the most important rock mass classification systems currently in use were considered. The aims of engineering classification of rock masses have been described by Bieniawski, (1984):

- To divide a particular rock mass into groups of similar behaviour;
- To provide a basis for understanding the characteristics of each group;
- To yield quantitative data for engineering design;
- To provide a common basis for communication;
According to the Bieniawski’s suggestion a rock mass classification system must have the following characteristics:

- Simple, easily remembered and understandable
- Each term must be clear and the terminology used must be widely understood by engineers and geologists
- The most significant properties of the rock masses must be included
- It must be based on measurable parameters which can be determined by relevant tests quickly and cheaply in the field
- It must be based on a rating system
- It must provide quantitative data for the design of rock support

Recently, many different rock mass classification systems have been developed for characterising rock from the viewpoint of excavation. Of the different rock mass classifications the Geomechanics Classification System and the Q System are more used for assessing the rock mass quality and rock slope stability analysis for surface mining. The more widely used classification systems are listed in Table 2.7. Most important of rock mass classifications are presented briefly in the following section:

2.10.2 Terzaghi Rock Mass Classification

This classification has been introduced by Terzahgi in 1946. The classification is one of the first systematic rock mass classifications used for evaluation of long time loads in tunnelling, but this classification is not suitable for modern tunnelling, especially in the shotcreted and bolted tunnels. In order to make the classification useful for tunnels, Terzaghi, (1946) proposed roof and side loads which might be expected on tunnel supports for each of the classification divisions (Farmer, 1983). Most important of rock mass classifications are presented in Table 2.8.

In this system the width and height of the tunnel excavation is considered as important factors during the tunnel life. This classification was used for a long period to estimate the load and design of appropriate support by steel arches in tunnelling.
## Table 2.8 Major engineering rock mass classification currently in use
(After Bieniawski, 1992).

<table>
<thead>
<tr>
<th>Classification systems</th>
<th>Concepts</th>
<th>Applications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terzaghi, 1946</td>
<td>Rock load concept</td>
<td>Tunnelling</td>
</tr>
<tr>
<td>Lauffer, 1958</td>
<td>Stand up time</td>
<td>Tunnelling</td>
</tr>
<tr>
<td>Rabcewicz, Pacher, 1964</td>
<td>NATM</td>
<td>Tunnelling</td>
</tr>
<tr>
<td>Deere, 1964</td>
<td>R Q D index</td>
<td>Core logging, tunnelling</td>
</tr>
<tr>
<td>Wickham et al, 1972</td>
<td>RSR system</td>
<td>Tunnelling</td>
</tr>
<tr>
<td>Bieniawski, 1973</td>
<td>RMR system</td>
<td>Underground and surface mining</td>
</tr>
<tr>
<td>Barton et al, 1974</td>
<td>Q System index</td>
<td>Underground and surface mining</td>
</tr>
<tr>
<td>Franklin, 1975</td>
<td>Strength -Size</td>
<td>Tunnelling</td>
</tr>
<tr>
<td>Laubscher et al, 1977</td>
<td>MRMR system</td>
<td>Coal mining</td>
</tr>
<tr>
<td>ISRM, 1981</td>
<td>Basic geotechnical classification</td>
<td>General communication</td>
</tr>
<tr>
<td>Ghose and Raju, 1981</td>
<td></td>
<td>Coal mining</td>
</tr>
<tr>
<td>Kendorski et al</td>
<td></td>
<td>Hard rock mining</td>
</tr>
<tr>
<td>Serafin and Pereira, 1983</td>
<td></td>
<td>Foundations</td>
</tr>
<tr>
<td>Gonzales de Vallejo, 1983</td>
<td></td>
<td>Tunnelling</td>
</tr>
<tr>
<td>Unai, 1983</td>
<td></td>
<td>Roof bolting/coal</td>
</tr>
<tr>
<td>Romana, 1985</td>
<td></td>
<td>Slope stability</td>
</tr>
<tr>
<td>Newman, 1985</td>
<td></td>
<td>Coal mining</td>
</tr>
<tr>
<td>Venkateswarlu, 1986</td>
<td></td>
<td>Coal mining</td>
</tr>
<tr>
<td>Robertson, 1988</td>
<td></td>
<td>Slope stability</td>
</tr>
</tbody>
</table>
2.10.3 Stand-up Time Classification

This classification was introduced by Lauffer in 1958. This classification has been used on unsupported tunnels. The concept of stand-up is the time for which the tunnel will be stable without any support. Stand-up time is very important in tunnelling, particularly in soft rocks. In fact there are relationship between rock mass quality and stand-up time of tunnel. In this classification Lauffer stated that an increase in tunnel span has influence on the reduction of the stand-up time. In addition this classification introduced that the concept of stand-up time and the span of the tunnel are important factors in the design of the type of tunnel support.

In this classification Lauffer has suggested that the stand-up time for any span of tunnels depends on rock mass characteristics and on the rock class. Lauffer defined seven classes for rock masses which class A relates to very good rock and class G is for very poor rock.

2.10.4 Rock Quality Designation (RQD)

This rock mass classification was introduced by Deere in 1964. In this system of classification, rock quality designation is the basis, and is a simple way for the description of rock mass quality, RQD obtained using recovered drill cores from diamond drilling. It depends on the orientation of joint sets, filling materials and length of fractures. This method is a more commonly used method of rock mass classification, particularly useful in classifying rock masses for engineering purposes.

Generally RQD is a quantitative index for the assessment of rock mass quality designation and it can be defined as the percentage of the core pieces which are more than 10 cm in length, divided to length of borehole. On the other words it can be expressed by

$$ R.Q.D = \frac{\text{length of core in pieces more than 10 cm}}{\text{length of run}} \times 100 \quad (2.7) $$

It should be noted that the diameter of the core should be 42 mm (BXM). It is obvious that the RQD of rock mass depends on some factors of rock mass, such as joint tightness, orientation of joints and orientation of drilling and kinds of filling materials.
Therefore RQD does not fully describe a rock mass character completely. However RQD can be used for tunnelling with narrow wide from 6 to 12 m span (Bieniawski, 1970). According to the value of the RQD obtained, the rock mass is divided into five categories. The different ranges of RQD are given in the Table 2.9.

Table 2.9 Various ranges of (RQD) rock quality designation, (After Deere, 1964).

<table>
<thead>
<tr>
<th>RQD. %</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;25</td>
<td>Very poor</td>
</tr>
<tr>
<td>25 - 50</td>
<td>Poor</td>
</tr>
<tr>
<td>50 - 75</td>
<td>Fair</td>
</tr>
<tr>
<td>75 - 90</td>
<td>Good</td>
</tr>
<tr>
<td>90 - 100</td>
<td>Excellent</td>
</tr>
</tbody>
</table>

Laubscher, (1990) suggested that in the determination of the RQD of rock surfaces, the sampling line must be likened to a borehole core and the following points observed:

- Experience in the determination of the RQD of core is necessary
- Do not be misled by blasting fractures
- Weaker bedding planes do not necessarily break when cored
- Assess the opposite wall where a joint forms the side-wall
- Shear zones greater than 1 m must be classified separately

2.10.5 Rock Structure Rating System (RSR)

The rock structure rating method was introduced by Wickham, Tiedemann and Skinner in 1972. In this system, numerical ratings and weightings are related for rock mass quality and excavation of tunnel and steel support requirements. This system presents a quantitative method for evaluating the quality of rock masses and designing the
appropriate support. The rock structure system was the first complete rock mass classification system proposed up to 1972.

The R.S.R. was a quantitative classification and also it was a rock mass classification which incorporates many parameters, such as

- Rock type.
- Rock hardness.
- Geological structure.
- Joint spacing.
- Joint orientation.
- Direction of tunnel drive.
- Joint condition.
- Amount of water inflow.

RSR is a complete classification with an input and output, therefore the RSR considers three groups of factors effective in rock mass behaviour:

- The effect of geological parameters including rock type, hardness and geological structures (fault, fold, bedding plane, etc).
- The effect of joint pattern and direction of tunnel drive (construction parameters)
- The effect of ground water and joint condition.

The RSR classification system was changed in 1974 and in this year the latter report represents the latest information available. The RSR classification system is a good method for selecting steel rib support in rock tunnels. This classification is not recommended for selection of rock bolt and shotcreted support, because each empirical method needs a range of sufficient and reliable data for developing it. Most of the input data would be normally included in the standard joint survey. Therefore the lack of definition such as slightly faulted, folded rock and moderately weathered may lead to some confusion (Bieniawski, 1984).
2.10.6 Rock Mass Rating (RMR)

The Geomechanics Classification System is based on the classification system proposed by Bieniawski (1973). This method underwent several changes in 1974, 1975, 1976, and 1979. The geomechanics classification was initially based on 49 case histories; of course, details of these cases and their relation have not been recommended to support the system. Generally, application of this method to mining has resulted in generation of a data base. The accuracy of the geomechanics classification depends on the method of sampling of the area being investigated. This principle applies to all classification investigations whether in a civil or a mining engineering situation.

Six principal parameters form the basis of the geomechanics classification system:

- Uniaxial compressive strength of intact rock.
- Rock quality designation (RQD.).
- Ground water condition.
- Spacing of discontinuities.
- Orientation of discontinuities.
- Condition of discontinuities.

The classification system known as the mining rock mass rating (MRMR) system was introduced in 1974 as a development of the CSIR geomechanics classification system. The development is based on the concept of in situ and adjusted ratings, the parameters and values being related to complex mining situations. Since 1974, the MRMR system has been modified and improved. This classification system has been used successfully in mining projects in Canada, Chile, Philippines, South Africa, USA, and Zimbabwe (Laubscher, 1990) and UK (Sunu, 1988; Gahrooei, 1989; Eksi, 1987 and El-Mherig, 1986).

In the assessment of rock mass behaviour in a mining environment, the rock mass rating (RMR) is adjusted for weathering, mining-induced stresses, joint orientation, and blasting effects. The adjusted rating is called the mining rock mass rating or MRMR. It is possible to use the rating to determine an empirical rock mass strength (RMS) in MPa. The in-situ rock mass strength (RMS) is adjusted as above to give a design rock
mass strength (DRMS). This is extremely useful when related to the stress environment, and has been used for mathematical modelling.

All of these factors are measurable in the field and can also be obtained from borehole data. It should be noted that the value of the discontinuity orientation is not given in quantitative terms, but by qualitative descriptions such as favourable or unfavourable. Each group of these parameters has different ranges of values which are given in the Table 2.10.

As a result the rock mass rating (RMR) is obtained by summing the individual parameter ratings:

\[ RMR = (1 + 2 + 3 + 4 + 5) - (b) \]

(2.8)

b is rating adjustment for discontinuity orientations.

Rock mass condition depends on ratings which are based on the six principle parameters. The best condition for the rock mass in engineering design is that a higher rating of classification is obtained. Therefore the importance of ratings which are given for discontinuity spacing to rock mass having three sets of discontinuities. Also determination of RMR, based on the characters of the discontinuities depends on the engineering application whether it be tunnel, slope or foundation. Of course orientation of discontinuities, such as strike and dip direction of joints is very important in these engineering applications. In this case description of joint orientation by Wickham, Tiedman and Skinner, (1972) will be sufficient for civil engineering projects.

Therefore, the importance of rock mass classification is to evaluate the major properties of the rock mass and to assign a numerical value to each parameter and to combine these values into a total classification rating value for the assessment of the rock mass quality. According to the geomechanics classification system, the rating of rock masses includes civil engineering project five groups of rock mass classes.
Table 2.19 Geomechanics Classification of Jointed Rock Masses

### A: Classification Parameters and Their Ratings

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Ranges of Values</th>
<th>For this low range uniaxial compressive test is preferred</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength of intact rock material</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uniaxial compressive strength</td>
<td>&gt; 250 Mpa</td>
<td>5-25 Mpa</td>
</tr>
<tr>
<td></td>
<td>100-250 Mpa</td>
<td>1-5 Mpa</td>
</tr>
<tr>
<td></td>
<td>50-100 Mpa</td>
<td>&lt;1 Mpa</td>
</tr>
<tr>
<td></td>
<td>25-50 Mpa</td>
<td></td>
</tr>
<tr>
<td>Rating</td>
<td>15</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>0</td>
</tr>
<tr>
<td>Drill Core quality (ROD)</td>
<td>90% - 100%</td>
<td>&lt; 25%</td>
</tr>
<tr>
<td>Rating</td>
<td>20%</td>
<td>3</td>
</tr>
<tr>
<td>Spacing of discontinuities</td>
<td>&gt; 2 m</td>
<td></td>
</tr>
<tr>
<td>Rating</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>Condition of discontinuities</td>
<td>Very rough surfaces</td>
<td></td>
</tr>
<tr>
<td>Rating</td>
<td>30</td>
<td>0</td>
</tr>
</tbody>
</table>

### B: RATING ADJUSTMENT FOR JOINT ORIENTATION

<table>
<thead>
<tr>
<th>Strike and Dip Orientations of Joints</th>
<th>Very favourable</th>
<th>Favourable</th>
<th>Fair</th>
<th>Unfavourable</th>
<th>Very Unfavourable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnels</td>
<td>0</td>
<td>-2</td>
<td>-5</td>
<td>-10</td>
<td>-12</td>
</tr>
<tr>
<td>Foundations</td>
<td>0</td>
<td>-2</td>
<td>-7</td>
<td>-15</td>
<td>-25</td>
</tr>
<tr>
<td>Slopes</td>
<td>0</td>
<td>-5</td>
<td>-25</td>
<td>-50</td>
<td>-60</td>
</tr>
</tbody>
</table>

### C: ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

<table>
<thead>
<tr>
<th>Rating</th>
<th>Class No</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 - 81</td>
<td>I</td>
<td>Very good rock</td>
</tr>
<tr>
<td>80 - 41</td>
<td>II</td>
<td>Good rock</td>
</tr>
<tr>
<td>40 - 21</td>
<td>III</td>
<td>Fair rock</td>
</tr>
<tr>
<td>&lt; 20</td>
<td>IV</td>
<td>Poor rock</td>
</tr>
<tr>
<td>&lt; 20</td>
<td>V</td>
<td>Very poor rock</td>
</tr>
</tbody>
</table>

### D: MEANING OF ROCK MASS CLASSES

<table>
<thead>
<tr>
<th>Class No</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average stand-up time</td>
<td>10 Years</td>
<td>6 Months</td>
<td>1 Week</td>
<td>10 hours</td>
<td>30 minutes</td>
</tr>
<tr>
<td>Cohesion of the rock mass</td>
<td>&gt; 400 Kpa</td>
<td>300 - 400 Kpa</td>
<td>200 - 300 Kpa</td>
<td>100 - 200 Kpa</td>
<td>&lt; 100 Kpa</td>
</tr>
<tr>
<td>Friction angle of the rock mass</td>
<td>&gt; 45°</td>
<td>35° - 45°</td>
<td>25° - 35°</td>
<td>15° - 25°</td>
<td>&lt; 15°</td>
</tr>
</tbody>
</table>

### E: The Effect of Joint Strike and Dip Orientation in Tunneling

<table>
<thead>
<tr>
<th>Strike perpendicular to tunnel axis</th>
<th>Strike parallel to tunnel axis</th>
<th>Dip 0° - 20°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drive with dip</td>
<td>Drive against dip</td>
<td>Irrespective of strike</td>
</tr>
<tr>
<td>Dip 45° - 90°</td>
<td>Dip 20° - 45°</td>
<td>Dip 45° - 90°</td>
</tr>
<tr>
<td>Very favourable</td>
<td>Favourable</td>
<td>Fair</td>
</tr>
<tr>
<td></td>
<td>Fair</td>
<td>Unfavourable</td>
</tr>
</tbody>
</table>
Moreover the rock masses are in groups of twenty ratings each. Different ratings of RMR classes are shown in Table 2.11.

Table 2.11 Illustration of RMR classes

<table>
<thead>
<tr>
<th>Rating</th>
<th>100-81</th>
<th>80-61</th>
<th>60-41</th>
<th>40-21</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class-no</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Description</td>
<td>very good rock</td>
<td>good rock</td>
<td>fair rock</td>
<td>poor rock</td>
<td>very poor rock</td>
</tr>
</tbody>
</table>

Since the various parameters are not equally important for the overall classification of a rock mass, importance ratings are allocated to the different value ranges of the parameters, and finally a higher rating indicates better rock mass condition. Some authors such as the following, have suggested different equation to correlate RMR and Q indexes:

\[
RMR = 19 \ln (Q) + 26 \quad (2.9) \quad \text{(Singh et al, 1986)}
\]
\[
RMR = 18.79 \ln (Q) + 13.48 \quad (2.10) \quad \text{(Sunu, 1988)}
\]
\[
RMR = 9 \ln (Q) + 44 \quad (2.11) \quad \text{(Bieniawaski, 1989)}
\]

2.10.7 Rock Mass Quality System (Q- system)

This system has been introduced on 212 case records in Scandinavia by Barton (1974). The classification of rock mass is based on six parameters which are used in the description of rock mass quality, and these are include:

- R.Q.D (rock quality designation).
- \( J_n \) = number of joint sets.
- \( J_r \) = rating of joint roughness.
- \( J_a \) = joint alteration number (degree of alteration).
- \( J_w \) = joint water reduction number.
- S.R.F. or stress reduction factor.
CHAPTER TWO  
Effects of Rock Mass Characteristics on Slope Stability

Generally the Q-system of rock mass classification is based on a numerical assessment of the rock mass quality. According to the values of each of six parameters the above, rock mass quality Q is as follows:

\[
Q = \frac{R.Q.D}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{S.R.F}
\]

(2.12)

In the above equation, there are three pairs of ratio, such as

\[
\left( \frac{R.Q.D}{J_n} \right), \left( \frac{J_r}{J_a} \right), \left( \frac{J_w}{S.R.F} \right)
\]

Each of the quotients represents an important quantity describing the rock mass, and defines its structure. The first term (RQD / Jn) is the relative block size of jointed blocks and the second division is (Jr / Ja) which is the shear strength of the block surface or inter block shear strength and the third section is (Jw / SRF) the active stress or environmental conditions influencing the behaviour of the rock mass. A description and rating of each parameter is given in the following:

* Ratings of RQD % parameter is illustrated in table 2.9.

* Description and ratings of the Jn parameter

<table>
<thead>
<tr>
<th>Rating</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>i</td>
<td>Massive</td>
<td>0.5-1</td>
</tr>
<tr>
<td>ii</td>
<td>One joint set</td>
<td>2</td>
</tr>
<tr>
<td>iii</td>
<td>One joint set plus random</td>
<td>3</td>
</tr>
<tr>
<td>iv</td>
<td>Two joint sets</td>
<td>4</td>
</tr>
<tr>
<td>v</td>
<td>Two joint sets plus random</td>
<td>6</td>
</tr>
<tr>
<td>vi</td>
<td>Three joint sets</td>
<td>9</td>
</tr>
<tr>
<td>vii</td>
<td>Three joint sets plus random</td>
<td>12</td>
</tr>
<tr>
<td>viii</td>
<td>Four or more joint sets</td>
<td>15</td>
</tr>
<tr>
<td>ix</td>
<td>Crushed rock, earthlike</td>
<td>20</td>
</tr>
<tr>
<td>Note:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>For intersection use (3.0 Jn)</td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>For portals use (2.0 Jn)</td>
<td></td>
</tr>
</tbody>
</table>

* Description and ratings for the parameter Jr.

<table>
<thead>
<tr>
<th>Rating</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>i</td>
<td>Discontinuous joints</td>
<td>4</td>
</tr>
<tr>
<td>ii</td>
<td>Rough or irregular, undulating</td>
<td>3</td>
</tr>
<tr>
<td>iii</td>
<td>Smooth undulating</td>
<td>2</td>
</tr>
<tr>
<td>iv</td>
<td>Slickensided, undulating</td>
<td>1.5</td>
</tr>
<tr>
<td>v</td>
<td>Rough or irregular, planer</td>
<td>1.5</td>
</tr>
<tr>
<td>vi</td>
<td>Smooth, planar</td>
<td>1.0</td>
</tr>
<tr>
<td>vii</td>
<td>Slickensided, planar</td>
<td>0.5</td>
</tr>
</tbody>
</table>
### CHAPTER TWO
Effects of Rock Mass Characteristics on Slope Stability

#### Description and ratings for the parameter $J_a$

<table>
<thead>
<tr>
<th>Description</th>
<th>$J_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>i  Tightly healed, hard, non-softening, impermeable</td>
<td>0.75</td>
</tr>
<tr>
<td>ii joint walls surface staining only</td>
<td>1.0</td>
</tr>
<tr>
<td>iii Slightly altered joint walls</td>
<td>2.0</td>
</tr>
<tr>
<td>iv Slightly or sandy clay coating, small clay fraction</td>
<td>3.0</td>
</tr>
<tr>
<td>v Softening or low friction clay mineral coating</td>
<td>4.0</td>
</tr>
<tr>
<td>vi Sandy particles, clay-free disintegrated rock</td>
<td>4.0</td>
</tr>
<tr>
<td>vii Strongly over-consolidated non-softening clay mineral fillings (continuous, but &lt;5 mm thickness)</td>
<td>6.0</td>
</tr>
<tr>
<td>viii Medium or low over-consolidation, softening clay mineral fillings (continuous but &lt;5 mm thickness)</td>
<td>8.0</td>
</tr>
<tr>
<td>ix Swelling clay filling, montmorillonite (continuous but &lt;5 mm thickness)</td>
<td>8-12</td>
</tr>
<tr>
<td>x Zones or bands of disintegrated crashed rock and clay (see G.H.F)</td>
<td>6.8 or 8-12</td>
</tr>
<tr>
<td>xi Thick, continuous zones or band of clay (see G.H.F)</td>
<td>10-13 or 13-20</td>
</tr>
</tbody>
</table>

#### Description and rating for the parameter $J_w$

<table>
<thead>
<tr>
<th>Description</th>
<th>$J_w$</th>
</tr>
</thead>
<tbody>
<tr>
<td>i  Dry excavation or minor inflow</td>
<td>1.0</td>
</tr>
<tr>
<td>ii Medium inflow or pressure occasional outwash of joint fillings.</td>
<td>0.66</td>
</tr>
<tr>
<td>iii Large inflow or high pressure in competent rock with unfilled joints</td>
<td>0.5</td>
</tr>
<tr>
<td>iv Large inflow or high pressure, considerable outwash of joint fillings</td>
<td>0.3</td>
</tr>
<tr>
<td>v Exceptionally high inflow or water pressure at blasting decaying with time.</td>
<td>0.2-0.1</td>
</tr>
<tr>
<td>vi Exceptional high inflow or water pressure continuing without noticeable decay.</td>
<td>0.1-0.05</td>
</tr>
</tbody>
</table>

#### Description and ratings for parameter S.R.F

<table>
<thead>
<tr>
<th>Description</th>
<th>(S.R.F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>i  Multiple occurrences of weakness zones containing clay or chemically disintegrated rock.</td>
<td>10</td>
</tr>
<tr>
<td>ii Single weakness zones containing clay or chemically disintegrated rock, depth of excavation &lt; 50 m</td>
<td>5.0</td>
</tr>
<tr>
<td>iii Single weakness zones containing clay or chemically disintegrated rock (depth of excavation &gt; 50 m)</td>
<td>2.5</td>
</tr>
<tr>
<td>iv Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)</td>
<td>7.5</td>
</tr>
<tr>
<td>v Single shear zones in competent rock (clay free), (depth of excavation &lt; 50 m)</td>
<td>5.0</td>
</tr>
<tr>
<td>vi Single shear zones in competent rock (clay free), (depth of excavation &gt; 50 m)</td>
<td>2.0</td>
</tr>
<tr>
<td>vii Loose open joints, heavily jointed or sugar cube etc (any depth)</td>
<td>5.0</td>
</tr>
</tbody>
</table>
The Q value relates to tunnel support requirements by defining the equivalent dimensions of the excavation. This equivalent dimension, which is a function of the size of the excavation is obtained by dividing by the span diameter to give the excavation support ratio (ESR.) Barton, (1974).

2.10.8 WEAKENING COEFFICIENT CLASSIFICATION SYSTEM

Weakening Coefficient is a classification system proposed by Singh (1986). This system was modified by the author to be employed as a rock mass classification in the design of rock slopes. This system was developed to define a reduction factor, weakening coefficient (WC) which could be applied to the intact samples values for determination of rock mass properties. This system of classification including the five geotechnical following parameters:

- Rock quality designation, (RQD)
- Joint spacing index, $K_1$
- Joint surface index (roughness), $K_2$
- Joint infilling materials index, $K_3$
- Joint aperture index, $K_4$

The parameters of weakening coefficient classification system and their indices and also rating adjustment for orientation of discontinuities are given in Table 2.12. These parameters take parts in calculation of the overall jointing coefficient can be calculated from the Equations 2.13 and 2.14 as follows:

$$K = K_1 \times K_2 \times K_3 \times K_4$$ \hspace{1cm} (2.13)

$$WC = K \times RQD$$ \hspace{1cm} (2.14)

where

WC is the weakening coefficient of the rock mass and RQD or R is the rock quality designation. It should be noted that a correlation has been carried out by Gahroooee (1989) between RMR values and the corresponding weakening coefficient and the following relation ship was proposed.

$$WC = 0.018 \times e^{(0.039RMR)}$$ \hspace{1cm} (2.15)
Table 2.12 Weakening Coefficient Classification System (Modified after Singh, 1986)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Very poor</th>
<th>Poor</th>
<th>Moderate</th>
<th>Good</th>
<th>Strong</th>
</tr>
</thead>
<tbody>
<tr>
<td>RQD</td>
<td>&lt;20</td>
<td>40</td>
<td>40 - 60</td>
<td>60 - 80</td>
<td>&gt;80</td>
</tr>
<tr>
<td>Index (R)</td>
<td>&lt;0.2</td>
<td>0.4</td>
<td>0.4 - 0.6</td>
<td>0.6 - 0.8</td>
<td>&gt;0.8</td>
</tr>
<tr>
<td>Discontinuity spacing</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Index (K_j)</td>
<td>0.6</td>
<td>0.7</td>
<td>0.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Joint surface</td>
<td>Slickensided</td>
<td>Polished</td>
<td>Smooth</td>
<td>Rough</td>
<td>Dormant</td>
</tr>
<tr>
<td>Index (K_j)</td>
<td>0.6</td>
<td>0.7</td>
<td>0.8</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>Joint infilling materials</td>
<td>Open (&gt;5 mm)</td>
<td>Open (&lt;5 mm)</td>
<td>Soft filling</td>
<td>Tight filling</td>
<td>Aspirate</td>
</tr>
<tr>
<td>Index (K_j)</td>
<td>0.6</td>
<td>0.7</td>
<td>0.8</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>Joint aperture</td>
<td>&gt;5 mm</td>
<td>5 mm</td>
<td>1 - 5 mm</td>
<td>0.1 - 1.0 mm</td>
<td>&lt;0.1</td>
</tr>
<tr>
<td>Index (K_j)</td>
<td>0.5</td>
<td>0.6</td>
<td>0.7</td>
<td>0.8</td>
<td>0.9</td>
</tr>
<tr>
<td>Weakening</td>
<td>&lt;0.04</td>
<td>0.04 - 0.1</td>
<td>0.1 - 0.2</td>
<td>0.2 - 0.5</td>
<td>0.5 - 1.0</td>
</tr>
<tr>
<td>Coefficient (WC)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Rating Adjustment for Discontinuity Orientation

<table>
<thead>
<tr>
<th>Discontinuity orientation</th>
<th>No modes of failure</th>
<th>Potential mode of failure</th>
<th>One mode of failure</th>
<th>Two modes of failure</th>
<th>Several modes of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope</td>
<td>WC×1</td>
<td>WC×0.833</td>
<td>WC×0.37</td>
<td>WC×0.135</td>
<td>WC×0.1</td>
</tr>
</tbody>
</table>
The weakening coefficient classification system can be used for determination of the constants $m$ and $s$ of the Hoek and Brown criterion. These constants parameters can be determined by the empirical equations propose by Gahrooee (1989) as follows:

\[
\text{Log (RQD)} = 0.118 + 1.827 \times \text{Log (WC)} \tag{2.16}
\]

\[
\text{Log (K_2)} = 0.047 + 4.052 \times \text{Log (WC)} \tag{2.17}
\]

\[
m = m_i \times K_1 \tag{2.18}
\]

\[
s = s_i \times K_2 \tag{2.19}
\]

where;

$WC =$ weakening coefficient of the rock mass

$m$ and $s =$ constants for the rock mass in the Hoek and Brown criterion

$m_i$ and $s_i =$ constants for the intact rock in the Hoek and Brown criterion

### 2.11 CONCLUSIONS

Due to the presence of structural features and discontinuities within the rock masses, the behaviour of the rock masses are mainly controlled by these structural features. Therefore, it is essential to consider the discontinuity characteristics of rock masses at the first step of the investigations in each rock engineering project, particularly in open pit mining.

This chapter consists of a literature review concerned with the geological data acquisition in assessing the stability of heavily jointed rock masses. The discontinuity characteristics and factor affecting the stability of rock slopes in surface mining were reviewed and the methods of data collection were considered. In this literature review it was found that the frequency distribution of discontinuity characteristics becomes an important factor affecting the stability of rock slopes. It should be noted that the data collected to be as representative of rock mass and the accuracy of the data is highly depend upon the methods of data collection. Different methods of rock mass classifications and their applications were reviewed and it was found that some of rock mass classifications such as RMR and WC (Weakening Coefficient classification) are more suitable for investigating the quality of the rock mass for the aims of open pit mining.
Chapter 3
Data Acquisition For Estimation of Rock Mass Strength from Laboratory Testing
CHAPTER THREE

DATA ACQUISITION FOR ESTIMATION OF ROCK MASS STRENGTH FROM LABORATORY TESTING

3.1 INTRODUCTION

Variations in the measurement of rock properties and inherent characteristics of rock are due not only to the physical dimensions of the laboratory specimens used, but also the testing equipment, test techniques, rate of loading, and rate of displacement. In order to control these effects, the International Society for Rock Mechanics (ISRM) commission on Standardisation of Laboratory and Field Testing was established in 1965. A careful consideration of the solutions to a number of practical engineering problems suggests that the rock mechanics laboratory tests play a relatively minor role in the range of factors to be considered (Hoek, 1977).

In the last three decades considerable progress has been made on the in-situ testing of rock masses. The results have shown that the mechanical behaviour of particular types of rock can vary from place to place. Bearing this in mind, an effort should be made to study the mechanical properties of rock masses through an in-situ testing program wherever possible. Lama and Vutukuri (1978) state that the first consideration in studying the properties of rock masses should be to determine the characteristics of the discontinuities, and also the deformability of a rock mass and its anisotropy results, predominantly from the displacements of the unit elements composing the structure of the rock mass. The strength of a rock mass is in fact its residual strength which, together with its anisotropy, is governed by the interlocking bonds of the unit "elements" forming the rock mass.
The intact rock properties have a great influence on the behaviour of rock masses; particularly in rock slope stability analyses. The strength and deformation properties of intact rock material are affected by many factors. Bieniawski, (1984) lists the most important of these factors. The factors are; anisotropy, moisture content, pour water pressure, confining pressure, time - dependent deformation, rate of loading and the size and shape of the specimen. Efficient surface mining operations can only be achieved by obtaining a clear definition of the engineering properties of the rock mass. The prediction of rock mass behaviour is based on site investigation, rock mass condition, test results, field data analysis scale effects and, consequently, the operating stress field.

In the last three decades, mechanical properties of rocks and rock masses have been studied by many authors (Goodman, 1974; Szlavin, 1974; Hoek, (1977); Holzhausen and Johnson, (1979); Beacher and Einstein, (1977); ISRM, (1977, 78, 81, 83, and 85); Price and Farmer, (1980); Stimpson, (1980, 1981); Ajtmatov and Mansurov, (1990); Sun et al (1990); Obara and Sugawara, (1990); Felice et al, (1991); Yutian et al, (1991); Archambault et al, (1992); Farmer, (1992); Holt and Kenter, (1992); Ghafoori et al, (1992); Zhang and Peng, (1993); Vutukuri, (1994) and Vutukuri and Hossaini,(1992).

The mechanical behaviour of rock masses cannot be determined, with any degree of accuracy, by laboratory testing because laboratory samples are fragments taken from intact rock and therefore, they cannot be truly representative of the rock mass. In this regard Lama and Vutukuri (1978,) believe that large scale in situ tests form an extremely important aspect in the design considerations of all major projects. They state that large scale in situ tests can be divided into three main categories including; deformability tests, shear tests and strength tests.

The consideration of the shear strength of discontinuities is an essential factor in the design of surface mines because the mechanical behaviour of rock samples is mainly governed by the shear displacement of joints, bedding planes and faults. In other words, the movement of rock blocks is resisted by the shear strength of the rocks. In surface mining, the composition of rock properties affects not only the rock slope's stability but
it also contributes to the selection of the method of extraction, the drilling method, and choices of equipment and blasting techniques.

In order to consider the properties of intact rock and the shear strength of jointed rock experiments based on laboratory and field studies were carried out on different types of quarries (porphyry, basalt and limestone samples) in order to ascertain the physical and mechanical properties of different types of rock and their effects on rock mass behaviour. Moreover, representative rock samples have been collected for laboratory testing and examination of their mineralogical compositions.

This chapter looks at methods of sampling, preparation of samples, and the laboratory testing of intact rock. Also considered are the shear strength parameters of joints and discontinuities in relation to the design of rock slope faces within a rock mass. It is assumed that the efficient stability design of surface mines, in part depends on obtaining an appropriate definition of the engineering properties of the intact rock and rock mass. An attempt is then made to determine the relationship between rock and rock mass properties

3.2 LABORATORY TESTING PROGRAM

Laboratory tests to study the behaviour of intact rock should be conducted in sequence. The importance of fundamental studies can not be overemphasised, particularly in regards to assessing rock slope stability and obtaining a comprehensive understanding of the intrinsic and mechanical properties of rock. A range of rock mass characteristics can be influenced by the properties of intact rock, in conjunction with the shear strength of the discontinuities. Such characteristics can also be used in rock slope stability analyses.

In general, the goal of geological and geotechnical investigations in rock engineering is to obtain sufficient information for design purposes from the strength and deformation properties of rocks and rock masses. This information provides the basis for safe and economic design of engineering constructions in rock masses. When considering the stability of slopes in a very heavily jointed rock mass, knowledge of the shear strength of the rock is required for use in failure type analysis (Hoek, 1977). Rock tests, particularly
in situ tests, are rather expensive, and it is desirable that we develop our understanding of rock behaviour to such an extent that, as far as possible, our predictions can be based on a few carefully selected and less expensive tests (Lama and Vutukuri, 1978). A major part of this study deals with uniaxial compressive strength, triaxial compressive strength, direct shear test, point load strength index, the tensile test, the moisture content, and bulk unit weight. This research has also attempted to establish the relationship between these parameters. Each direct shear test provides the following parameters:

- Normal stress
- Shear stress
- Normal displacement
- Shear displacement

The data which will be obtained from these parameters can lead to the construction of a shear stress - normal stress envelope, and a shear displacement - shear stress diagram. It can be used for the determination of the discontinuity friction angle and cohesive strength. The effect of intact rock properties on rock slope stability have been considered by many researchers, including Hoek and Brown, (1980); Hoek and Bray, (1981); Gray, (1988); Hoek, (1982); Datir et al, (1985); Singh et al, (1985); Singh and El-Mherig, (1985); Brawner et al, (1986); El-Mherig, (1986); Eksi, (1987); Sunu, (1988); Zhang and Peng, (1988); Gahrooeie, (1989); Franklin and Dusseault, (1989); Wittke, (1990); Giani, (1992); and Brady and Brown, (1993). The most important of these properties can be listed as follows:

- Uniaxial compressive strength
- Tensile strength
- Shear strength
- Elasticity modulus
- Modulus ratio
- Poisson’s ratio
- Porosity
- Density
• Water content

The results obtained from the assessment of engineering properties and rock behaviour based on laboratory or field testing can be used in rock slope stability analyses. The dimensions of core samples used in laboratory testing program are summarized in Table 3.1.

Table 3.1 Dimensions of NX samples for different tests.

<table>
<thead>
<tr>
<th>Type of tests</th>
<th>Sample length (mm)</th>
<th>L/D Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial Compressive</td>
<td>115 - 130</td>
<td>2.2 - 2.5</td>
</tr>
<tr>
<td>Triaxial Compressive</td>
<td>110 - 116</td>
<td>2.03 - 2.15</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>109 - 115</td>
<td>2.01 - 2.13</td>
</tr>
<tr>
<td>Point-Load (axial)</td>
<td>41.6 - 46.1</td>
<td>0.77 - 0.84</td>
</tr>
<tr>
<td>Point-Load (diametral)</td>
<td>58 - 71.3</td>
<td>1.09 - 1.30</td>
</tr>
<tr>
<td>Brazilian</td>
<td>28 - 42</td>
<td>0.52 - 0.77</td>
</tr>
</tbody>
</table>

3.3 SAMPLING AND METHODS OF PREPARATION OF THE SPECIMENS

The sampling of rock from surface mines is very important to ensure relevant laboratory testing results. Therefore, the samples should be representative of the rock mass. In general, there are three different methods of rock sampling in field investigations:

• Diamond core drilling method

• Core sampling from discontinuities in the field

• Block sampling in the laboratory

Since the total cost of sample preparation of block specimens is less than the cost of the other methods in the laboratory, the block sampling method was chosen throughout this research. The selected samples from the sites were marked to indicate the in-situ position. The average dimension of the block samples chosen is 50 x 30 x 25 cm. In the case of the shear strength test several naturally jointed core samples were prepared. The two halves of the rock samples containing a discontinuity were matched together by wire
loops so as to be representative of the natural joints in the rock mass. Details of this work are further described in chapter 4.

3.3.1 Specimen Preparation Methods

Because of the thickness of the block samples, and based on earlier experiments with coring of rock, drilling was carried out parallel to the discontinuities to obtain a higher percentage of core recovery. In the case of direct shear samples drilling was normal to the discontinuities so that the samples contained natural joints. In order to obtain core samples from the blocks of rock, it was necessary to cast the blocks in a casting material. Rock pattern plaster was used for this purpose which has a high degree of strength. One of the major problems of the drilling operation is vibration of the samples during drilling, which can lead to ineffective core recovery, irregular samples and unsafe operation of the drilling machine.

The most important advantages of casting the blocks of rock can be listed as follows:

- Increasing the safety of the drilling operation
- Obtaining good quality core samples and improving the yield
- To obtain core samples in the interest orientation, particularly for anisotropic rocks

For casting the block samples a suitable mould was used. The mould consisted of four steel side panels of 62 x 33 cm dimensions. The mould was flexible and adjustable for different sizes and shapes of rock samples. The side panels could be fixed to each other by clamps. Figures 3.1 and 3.2 shows the specimens which are cast in concrete. It should be noted that a mixture of five parts of aggregate (two parts of coarse grain and three parts of fine grains) together with four parts of sand and four parts of cement were used for casting the block samples before drilling. The materials for making the concrete were mixed using an electrical mixer.
Figure 3.1 Illustration of flexible mould used in casting of the block samples.

Figure 3.2 Illustration of samples cast in concrete.

Figure 3.2 Illustration of samples cast in concrete.
3.3.2 The Coring Machine Specification

The cast rock samples were drilled using a radial arm drilling machine in the departmental laboratory. This drilling machine is a Chinese Z-J model Z 3032 x 10 (I) which was designed for use in a mechanical engineering workshop. Figure 3.3 shows the Z-J drilling machine used for preparation of core samples. The main specifications of this drilling machine are given in Table 3.2.

Table 3.2 Specification of the Z-J drilling machine

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drilling diameters (mm)</td>
<td>54 and 38</td>
</tr>
<tr>
<td>Elevating speed of arm; m/min</td>
<td>1.27</td>
</tr>
<tr>
<td>Maximum permissible torque of spindle; N/m</td>
<td>196</td>
</tr>
<tr>
<td>Maximum permissible feed pressure of spindle; N</td>
<td>9800</td>
</tr>
<tr>
<td>Number of spindle feeds (mm/r)</td>
<td>8</td>
</tr>
<tr>
<td>Number of spindle speeds; (Step)</td>
<td>16</td>
</tr>
<tr>
<td>Power of the main drive motor; (KW)</td>
<td>2.2</td>
</tr>
<tr>
<td>Power of the arm elevating motor; (KW)</td>
<td>0.75</td>
</tr>
<tr>
<td>Power of the column clamping motor; (KW)</td>
<td>0.37</td>
</tr>
<tr>
<td>Power of the coolant pump; (KW)</td>
<td>0.09</td>
</tr>
<tr>
<td>Range of spindle speeds; (Rpm)</td>
<td>32 - 2500</td>
</tr>
<tr>
<td>Range of spindle feeds; (mm/r)</td>
<td>0.10 - 1.25</td>
</tr>
<tr>
<td>Swivel angle of arm</td>
<td>360°</td>
</tr>
<tr>
<td>Taper in spindle; (Morse No)</td>
<td>4</td>
</tr>
<tr>
<td>Weight of the machine; (kg)</td>
<td>2000</td>
</tr>
</tbody>
</table>
It is possible to adjust the speed and load of the drilling machine by the use of two knobs within a range of 32 to 2500 rpm and 0.10 to 1.25 mm/r feed rate. Adjusting these ranges depends on the type of rock and it is generally necessary to determine these values experimentally. At the start of drilling great care must be taken to make a very slow contact between the rock surface and core barrel using the manual feeding position, because there is a danger of sliding of the core barrel on the surface of the specimen and associated damage. After manual drilling a few millimetres it is better to use the drilling machine in automatic feed. It should also be noted that a speed of 400 rpm and a 0.25 mm/r feed rate were used during the automatic feeding stages throughout the drilling program for hard rock samples. During the drilling program two different sized core barrels were used, as follows:

- Standard concrete type diamond impregnated core drill; size 54 mm core diameter with 60 mm outside diameter and 450 mm length.
- Standard concrete type diamond impregnated core drill; size 38 mm core diameter with 44 mm outside diameter and 450 mm length.

### 3.3.3 Preparation of Cylindrical Rock Specimens

The core samples obtained were prepared to the standard required for testing. Therefore, the prepared specimens were checked to see if they met the standards suggested by ISRM, (1981). According to these recommendations the cylindrical specimens were prepared to a length to diameter ratio suitable for laboratory testing. A diamond disc saw machine was used to cut the bottom off the specimens. In order to grind the ends of the specimens a mechanical surface grinding machine modified for grinding rock specimens was used. Figure 3.4 illustrates the grinding apparatus used.

It should be noted that ISRM, (1981) recommendations for tolerances of cylindrical specimens are as follows;

- Smoothness of the sides of the specimens to 0.3 mm
- Flatness of the ends of the specimens to 0.025 mm
- Perpendicularity of the ends to the axis of the specimen 0.058° or 0.001 radian
CHAPTER THREE
Data Acquisition and Estimation of Rock Mass Strength

Figure 3.3 Coring machine used for preparation of cylindrical specimens

Figure 3.4 Illustration of grinding machine used for grinding the ends of specimens
3.4 SIGNIFICANT FACTORS INFLUENCING STRENGTH OF ROCK AS DETERMINED IN THE LABORATORY

3.4.1 Intrinsic Factors Governing the Strength of Rock

(a) Density

Density is one of the intrinsic physical characteristics of rocks and it can be defined as the unit weight of the rock. Density has a very close relationship with the lithology of the rock. Dry and saturated densities were measured for different types of rock from different quarries (Porphyry, Basalt and Limestone). The results are shown in Table 3.3

(b) Porosity

Porosity identifies the ratio of the pore volume to the total volume and it expresses the proportion of void space in the rock sample. The most common method for measuring interconnected pore volume is to vacuum-saturate and space-dry the specimens, weigh them, oven-dry them, and weigh them again (Franklin and Dusseault, 1989). Porosity is an important intrinsic characteristic of rocks and has a great influence on the physical and mechanical behaviour of rocks. Some properties of rock such as its uniaxial compressive strength and its elasticity modulus are governed by porosity. The porosity of rocks can be described in categories such as dense, porous, and very porous. Generally low density rocks have porous structures.

(c) Moisture content

The determination of the water content is based on the measurement of the mass of water contained in a rock sample as a percentage of the oven-dried sample mass. The water content of a rock mass plays a great role in the behaviour of the rock mass particularly in respect to slope stability problems. In general, specimens which are moist have low strength in comparison with dry specimens. A digital balance with an accuracy of 0.001 grams was used for measuring the mass of the samples. Rock core samples previously prepared were used for this purpose.
Table 3.3 Results of water content and density tests for different type of rocks

<table>
<thead>
<tr>
<th>Type of rock</th>
<th>( m_1 ) (gr)</th>
<th>( m_2 ) (gr)</th>
<th>( M_s ) (gr)</th>
<th>( M_w ) (gr)</th>
<th>W%</th>
<th>psat (kg/m(^3))</th>
<th>( \rho_d ) (kg/m(^3))</th>
<th>( \rho ) (kg/m(^3))</th>
<th>( V_v ) (m(^3))</th>
<th>Msat (kg)</th>
<th>( V ) (m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porphyry section A and B (weathered)</td>
<td>628.125</td>
<td>624.475</td>
<td>604.475</td>
<td>560.475</td>
<td>3.65</td>
<td>0.603</td>
<td>2632.897</td>
<td>2611.43</td>
<td>2627.207</td>
<td>4.97E-6</td>
<td>0.60945</td>
</tr>
<tr>
<td>Porphyry section C and D (nonweathered)</td>
<td>650.1</td>
<td>648.15</td>
<td>628.15</td>
<td>560.15</td>
<td>1.95</td>
<td>0.31</td>
<td>2669.76</td>
<td>2660.99</td>
<td>2669.23</td>
<td>2.07E-6</td>
<td>0.63022</td>
</tr>
<tr>
<td>Limestone Marulan Quarry</td>
<td>426.271</td>
<td>425.817</td>
<td>405.817</td>
<td>405.817</td>
<td>0.454</td>
<td>0.11</td>
<td>2663.360</td>
<td>2657.087</td>
<td>2660.06</td>
<td>2.0E-6</td>
<td>0.406775</td>
</tr>
<tr>
<td>Basalt Dunmore Quarry</td>
<td>465.795</td>
<td>464.462</td>
<td>444.642</td>
<td>444.642</td>
<td>1.153</td>
<td>0.26</td>
<td>2714.754</td>
<td>2703.155</td>
<td>2710.167</td>
<td>1.9E-6</td>
<td>0.44655</td>
</tr>
</tbody>
</table>
The amount of water content of different types of rock (Porphyry, Basalt, and Limestone) was measured for moisture content by using the ISRM suggested method (1981) and the results are presented in the Table 3.3.

Where:

\[ m_1 = \text{the mass of container and the moist rock sample, g} \]
\[ m_2 = \text{the mass of container and the dried rock sample, g} \]
\[ M_w = \text{the mass of water, g} \]
\[ M_s = \text{the mass of solid particles, g} \]
\[ M_{sat} = \text{the saturated sample, surfaces dry mass, kg} \]
\[ V = \text{the Volume of specimen, m}^3 \]
\[ \rho_{sat} = \text{the saturated density, kg/m}^3 \]
\[ \rho_d = \text{the dry density, kg/m}^3 \]
\[ \rho = \text{the bulk density, kg/m}^3 \]

3.4.2 Extrinsic Factors Influencing Rock Strength

3.4.2.1 Specimen volume

It has been observed, through Experimentation, that in similar specimen geometry, the uniaxial compressive strength of rock material (UCS) varies with the specimen volume (Brady and Brown, 1993). The diameter of the specimen should be related to the size of the largest grain in the rock and it should be at least 10 times greater. It has been observed that the uniaxial compressive strength decreases with an increase in the volume of the specimen.

3.4.2.2 Strain rate

The observed behaviour of rock is not significantly influenced by varying the strain rate within the range that is convenient to use in quasi-static laboratory compression tests (Brady and Brown, 1993) while a loading rate of 0.5 - 1.0 MPa/s is recommended by
ISRM (1981) for use in the uniaxial compression test, and an axial strain rate of $10^5$ to $10^4$/s is also recommended by ISRM (1981). A variation in the strain rate or stress rate has an effect on the determination of the uniaxial compressive strength of rock. According to an ISRM (1981) suggestion, a routine test takes no more than 10 minutes and usually less. Throughout the course of this research, laboratory tests have been carried out according to ISRM's (1981) suggested practice.

3.4.2.3 Testing machine

Standard techniques for determination of the uniaxial compressive strength and deformability of rock samples have been suggested by the International Society for Rock Mechanics Commission on Standardisation of Laboratory and Field Testing. Whether or not the post-peak portion of the stress-strain curve can be followed and the associated progressive disintegration observed depends on the relative stiffness of the specimen and the testing machine. The type of testing machine is important in assessing the post failure behaviour of the rock samples. The post failure behaviour and the residual strength of the rock samples can be considered when using a servo controlled testing machine. A (Schenck Trebel) servo controlled testing machine was used for the laboratory testing program.

3.4.2.4 End effects and the influence of length to diameter ratio

According to ISRM (1981), the end of the specimen should be flat to 0.02 mm and should not depart from perpendicularity to the axis of the specimen by more than 0.001 radian or 0.05 mm in 50 mm. The end condition affects the uniaxial compressive strength as the stress distribution will vary throughout the specimen based on the specimen geometry. ISRM (1981) further suggests that the test specimens should be straight cylinders with a height to diameter ratio of 2.5 - 3.0 and a diameter preferably of not less than 54 mm (NX). The flatness of the ends of the specimens and also the height to diameter ratio have major effects on the strength of the rock samples. The uniaxial compressive strength is increased with a decrease in the height to diameter ratio.
3.5 ESTIMATION OF UNIAXIAL COMPRESSIVE STRENGTH OF ROCKS

The uniaxial compression test is mainly used for strength characterisation of intact rock samples. In concept, the uniaxial compressive strength is the maximum resistance of the intact rock sample under loading. The standards for this test were set up by the ISRM commission (1981) on the standardisation of laboratory and field tests. For many years the uniaxial, or unconfined, test was the main quantitative method for characterising the strength of rock material (Hawkes and Mellor, 1970). The uniaxial compression test is one of the most important tests in rock mechanics, because most of the engineering properties of rock samples can be determined from its results. A new method for measuring rock's complete stress-strain curve has been introduced by Yutian, et al, (1991). According to above researches the application of this method will be highly significant in researching the actual course of rock breaking and developing constitutive relationship under a different unloading rate. The following information can be derived from a complete stress - strain curve.

- Stress - strain relationship
- Post failure behaviour
- Maximum uniaxial compressive strength
- Elastic Modulus
- Poisson's ratio
- Stiffness

Different external and internal factors affect the uniaxial compressive strength of rock samples. For example the lithology ( mineralogy, grain size and porosity ) has a great influence on the uniaxial compressive strength of intact rock samples. The geometry, size, height to diameter ratio (h/d), rate of loading and surface condition are also important parameters which can influence the uniaxial compressive strength of intact rock samples. In the uniaxial test, a rock cylinder with a length two to three times its diameter is cut from a core using a diamond saw, and the ends are ground flat and perpendicular to the cylinder axis using a lapping machine. The test can be used only
when the core is available in suitable length, and when the rock is sufficiently sound to allow machining (Franklin and Dusseault, 1989).

The behaviour of rock specimens, which are short in relation to their diameter, is affected strongly by contact with the platens between which they are compressed. Even when the surfaces of the specimen and the platens are flat and parallel, the rigidity of the platens restricts the lateral expansion of the ends of the specimen. Singh and Eksi (1987) state that the strength of the specimen usually decreases with an increase in size and the size of the specimen also has a variable effect on the compressive strength of intact rock samples. The efficient stability of rock slopes in surface mining can only be achieved by clearly defining the engineering properties of the rock mass together with a good understanding of their likely interaction (Singh and Sunu, 1988). The advantages of the uniaxial compression test has been summarised by Broch and Franklin, (1972) as follows:

- The testing procedure is better known and evaluated.
- Results are variable for a wide variety of rock type, together with experience in linking these results to field performance.

In addition, they stated that there is a good correlation between the results of the uniaxial compression test and the diametral point load test. Their results showed a straight-line correlation whose slope of 27.3 corresponds to the ratio of uniaxial compressive strength to point load strength, averaged for the fifteen rock samples. Uniaxial compressive strength, the most commonly quoted index of rock behaviour, is size dependent. This result from the transfer of strain energy stored in a volume of rock to a fracture surface - as surface energy - at failure. It can be shown that the measured strength at failure is proportional to $L^{-1/2}$, where $L$ is a specimen dimension (Farmer, 1983).

A laboratory and field investigation program was carried out using samples from three producing quarries namely Mugga II, Marulan Limestone, and Dunmore. Additionally, representative block samples were taken from different parts of the sites for various laboratory tests and for examination of their mineralogy.
3.5.1 Sample Preparation for Uniaxial Compressive Strength Test

The procedure for the preparation of the specimens and testing according to the standards laid down by ISRM (1981) are as follows:

- Test specimens should be straight cylinders having a height to diameter ratio of 2.5 - 3.0
- Diameter of specimens preferably of not less than NX core size (approximately 54 mm). A height to diameter ratio of 2.0 was used for all compressive strength tests which were conducted on 54 mm diameter cores since the condition of the block samples did not allow for the preparation of longer specimens.
- The diameter of the test specimen should be measured to the nearest 0.1 mm by averaging two diameters measured at right angle and the length of the specimen shall be determined to the nearest 0.1 mm.
- Load on the specimen should be applied continuously at a constant stress rate such that failure will occur within 5 - 10 minutes of loading, alternatively the stress rate shall be within the limits of 0.5 - 1.0 MPa/sec.
- The number of specimens tested should be determined by practical consideration, but at least five are preferred.

Consequently a servo controlled testing machine (Schenck Trebel, 500 kN capacity and a semi - stiff machine (Instron Testing Machine, 1000 kN capacity) were used in this work and uniaxial compressive strength tests were carried out. One of the advantages of this type of machine is that its system allows study of the post failure behaviour of the rock samples. In this machine the force was measured either by oil pressure or by the movement of the digital transducers mounted to the upper cross head and functioning by the ring-torsion principle. The rate of displacement was controlled by the computer to the maximum displacement of the specimen.

All of the data relating to the maximum load, testing time and maximum displacement were recorded by an IBM micro computer monitoring system. Figures 3.5 and 3.6 shows the Schenck Trebel and Instron Testing machines used in the course of this research. The main specifications of the Schenck Trebel machine are given in Table 3.4.
3.5.2 Testing Procedure

Before testing commenced the specimens were carefully checked to ensure that they complied with the standards suggested by ISRM committee e.g straight cylinders having a height to diameter ratio of 2 to 2.5. The specifications of the samples were inputed into the computer. Additional information such as the testing time and the rate of loading were necessary for control of the testing process by the computer.

Table 3.4 Some of the main specifications of the Schenck Trebel Servo-controlled testing machine.

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal load</td>
<td>500 KN</td>
</tr>
<tr>
<td>Clearance between columns</td>
<td>700 x 2120 mm</td>
</tr>
<tr>
<td>Clearance between pressure columns</td>
<td>Max 350 mm</td>
</tr>
<tr>
<td>Piston stroke</td>
<td>200 mm</td>
</tr>
<tr>
<td>Machine base</td>
<td>1050 x 700 mm</td>
</tr>
<tr>
<td>Machine weight</td>
<td>5500 Kg</td>
</tr>
<tr>
<td>Machine height</td>
<td>2100 mm</td>
</tr>
<tr>
<td>Accuracy</td>
<td>1%</td>
</tr>
<tr>
<td>Hydraulic power pack</td>
<td>20 l/min</td>
</tr>
<tr>
<td>Oil pressure</td>
<td>Max 24 MPa</td>
</tr>
<tr>
<td>Pressure plates</td>
<td>520 x 320 x 100</td>
</tr>
</tbody>
</table>

The rate of displacement was within 100 μm/min was selected for rock samples. This rate of displacement was based on a maximum of 5 mm displacement during the test period. It should be noted that the maximum displacement for the specimens was between 3 to 5 mm failure which occurred in 10 to 15 minutes.

The maximum load on the specimens was recorded by computer with an accuracy of ± 0.01 KN. As mentioned above the load and deformation of the specimens were recorded at very short interval during the test by the use of three digital transducers which were mounted on the machine frame and recorded by the computer. The results of uniaxial compressive strength tests were presented in the form of stress - strain curves (Figures 3.7 to 3.10), and elastic modulus was calculated at a 50% of ultimate strength.
Figure 3.5  Illustration of Schenck Trebel, a servo controlled testing machine

Figure 3.6  Illustration of Instron testing machine, a semi-stiff machine
Figure 3.7 Stress - Strain Curve Uniaxial Compressive Strength Test (Sections A and B, Porphyry Rock Mugga II Quarry)
Figure 3.8 Stress - Strain Curve Uniaxial Compressive Strength Test (Sections C and D, Porphyry Rock Mugga II Quarry)
Figure 3.9 Stress - Strain Curve Uniaxial Compressive Strength Test (Limestone Samples from Marulan Quarry)
Figure 3.10 Stress - Strain Curve Uniaxial Compressive Strength Test (Basalt Samples from Dunmore Quarry)
The mechanical properties of the different type of rocks derived from uniaxial compressive strength tests are given in Table 3.5.

Table 3.5 Mechanical properties of the rocks resulted from uniaxial tests

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Young's Modulus</th>
<th>Poisson's ratio</th>
<th>UCS (peak)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$E_{(50)}$ (GPa)</td>
<td>$\nu$ (V)</td>
<td>(MPa)</td>
</tr>
<tr>
<td>Porphyry</td>
<td>Range</td>
<td>46 - 34.5</td>
<td>0.32 - 0.18</td>
</tr>
<tr>
<td>(N Weathered) (Sections A &amp; B)</td>
<td>Mean</td>
<td>38.5</td>
<td>0.22</td>
</tr>
<tr>
<td>Porphyry Weathered</td>
<td>Range</td>
<td>52.1 - 35.5</td>
<td>0.34 - 0.20</td>
</tr>
<tr>
<td>(Sections C &amp; D)</td>
<td>Mean</td>
<td>46.2</td>
<td>0.25</td>
</tr>
<tr>
<td>Limestone (Marulan Quarry)</td>
<td>Range</td>
<td>28 - 16.5</td>
<td>0.38 - 0.28</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td>18.45</td>
<td>0.35</td>
</tr>
<tr>
<td>Basalt (Dunmore Quarry)</td>
<td>Range</td>
<td>45.5 - 32.6</td>
<td>0.31 - 0.18</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td>39.3</td>
<td>0.24</td>
</tr>
</tbody>
</table>

3.5.3 Discussion on the Results of Uniaxial Compression Test

The uniaxial compression test, using cylinders of rock samples is the oldest and simplest test for determining the intact rock properties. The uniaxial compressive strength of the rock specimen is normally calculated from the ratio between the maximum load carried by the specimen in the test period and the cross-sectional area of the specimen in unit of MPa. It should be noted that an increase in cross-sectional area will occur during the test but, this slight increase in cross-sectional area of the specimen due to pre-rupture fracturing of the rock is unlikely to be an important consideration in strength calculations, particularly for brittle specimens like porphyry and basalt rocks. As can be gathered from Figures 3.7 to 3.10 the behaviour of the specimens are linear with a high range of Elastic modulus for the porphyry and basalt. A servo-controlled testing machine was used for the testing of specimens. However, because of the brittleness of the specimens it was not possible to control the post failure stress-strain behaviour of the rock samples and in the ultimate strength, the specimens burst instantaneously.
The values of Young's modulus and Poisson's rock ratio depend upon the stress applied to the rock. Up to this point, Poisson's ratio will increase with increasing stress up to its ultimate value (0.5) at the failure point. In other words, Young's modulus is dependant on the stiffness of the rock specimen. Therefore, Young's modulus and Poisson's ratio are not considered as a constant value but changes with level of stress. Hence, the application of their values in rock engineering should be related to the load applied to the structure.

3.6 DETERMINATION OF DEFORMATION PROPERTIES OF ROCK

3.6.1 Young's Modulus

Deformability of the rock specimen can be assessed by the uniaxial stress - strain test. In this method results are plotted on a typical uniaxial stress - strain curve. Young's Modulus or the Modulus of Elasticity can be obtained by the slope of the uniaxial stress - strain curve. During the initial application of the load, the rock progressively becomes denser and less porous, cracks, and joints also appear and as a result, it becomes stiffer and less deformable (Franklin, 1989). Determination of Young's Modulus and Poisson's Ratio requires longitudinal measurement and also diametral strain of the specimen should be established using transducers or strain gauges. The axial strain, $\varepsilon_a$, and diametrical strain, $\varepsilon_d$, may be recorded directly from strain indicating equipment or may be calculated from deformation depending upon the type of instrumentation. Axial strain can be calculated from the Equation 3.1

$$\varepsilon_a = \frac{\Delta l}{l_o} \quad (3.1)$$

where

$l_o$ = original measured axial length.

$\Delta l$ = change in measured axial length (defined to be positive for a decrease in length).

Diametrical strain may be determined either by the measuring the change in specimen diameter or by measuring the change in diameter, the diametral strain can be calculated from Equation 3.2
\[ \varepsilon_d = \Delta d / d_0 \] (3.2)

where;

\( d_0 \) = original undeformed diameter of the specimen.
\( \Delta d \) = change in diameter (defined to be negative for an increase in diameter).

The volumetric strain, \( \varepsilon_v \), for a given stress level can be calculated from the following equation (After ISRM, 1978).

\[ \varepsilon_v = \varepsilon_a + 2 \varepsilon_d \] (3.3)

Young's Modulus values were obtained by uniaxial compressive strength tests using the Schenck Trebel servo controlled testing machine. The value of Young's Modulus was calculated automatically based on the following equation at 50\% of the specimen's ultimate stress.

\[ E = \Delta \sigma / \Delta \varepsilon_a \] (3.4)

Modulus of Elasticity (E) was determined from the slope of the stress-strain curves from Figures 3.7 to 3.10. The results are given in Table 3.5.

3.6.2 Poisson's Ratio

Poisson's ratio can be defined as the ratio of the lateral (diametral) to the longitudinal (axial) strain. At low stress levels the lateral strain is usually less than one-quarter of the measured longitudinal strain and, at stresses up to about 50 - 60 \% of maximum uniaxial compressive strength, a Poisson's ratio range of 0.1 - 0.25 is typical. At higher stress levels approaching rupture longitudinal cracks appear that result in lateral strains much greater than those occurring longitudinally. At this stage Poisson's ratio increases dramatically (Franklin, 1989). Poisson's ratio may be calculated from the ISRM (1978) recommendation as follows:

\[ \theta = \frac{\text{slope of axial stress-strain curve}}{\text{slope of diametric stress-strain curve}} \]
Poisson's ratio can also be determined using the following equation.

\[
\vartheta = \frac{\Delta D / D}{\Delta L / L}
\]  

(3.5)

In the case of purely elastic deformation, the value of Poisson's ratio (\(\vartheta\)) cannot be higher than 0.5 which is the theoretical maximum. As mentioned in section 3.1, the engineering properties of the specimens obtained by laboratory or field testing depend upon a number of external factors. The value of Poisson's ratio so measured are affected by a number of factors such as the stress level, presence and absence of cracks, temperature, rate of loading etc (Lama and Vutukuri, 1987, Vol2). The Poisson's ratio (\(\vartheta\)) was measurable for the different types of rock by using a semi-stiff machine (Instron Testing Machine).

The Instron digital servo hydraulic testing machine is a hydraulic machine consisting of three main parts: The load frame, a hydraulic power pack and an electronic control console. The stiffness of the load frame is greater than the stiffness of the conventional screw driving system and it controls the stiffness of the load frame.

The rate of loading of 1.6 kN/s was set up and kept constant during testing. Since the specimens were of high strength and very brittle, when they reached peak strength and burst, it was not possible to obtain the post failure stress-strain behaviour of the rock samples. Generally, this phenomenon is due to the release of elastic energy stored in the machine during the loading period. This problem can be solved by using a servo controlled testing machine, or stiff machine. The rate of displacement is kept at a constant value and the load is increased to the peak strength (Uniaxial Compressive Strength) of the specimen.

The accuracy of the load measurement was ±10 N and for displacement was ±0.01 mm. The displacement rate was set up within the range 100 to 130 \(\mu\)m/min with the aim of carrying out the test within 10 -15 minutes. In order to determine the Poisson ratio of the specimens, two strain gauges were mounted on each specimen, one vertically and the other horizontally. It should be noted that, in using the strain gauges, a cautious
approach was taken in preparing the specimens for testing and measuring Poisson's ratio in order to avoid the presence of occluded air under the strain gauges, otherwise the results would have been less precise. Poisson's ratio for different type of rocks is given in Table 3.5.

3.7. DETERMINATION OF THE SHEAR STRENGTH OF ROCK BY DIRECT SHEAR TEST

3.7.1 Testing Procedures

The main aim of this test is to describe the most important engineering properties of joints relating to the evaluation of the shear strength of rocks and discontinuities. The direct shear test is one of the most commonly used methods for determining the shear strength of rocks and rock joints. In order to study the mechanical behaviour of joints several block samples containing natural joint sets were chosen from different open pit mines. Samples containing natural joints were prepared in the laboratory and tested using the direct shear box. Details of the shear strength of rock joints are described in chapter 4.

3.7.2 Determination of Friction Angle and Cohesion Factor

The determination of the friction angle ($\phi$) and cohesion factor ($C$) is the main aim of the direct shear test of discontinuities. From the Mohr - Coulomb criterion, the shear strength parameters, $C$ and $\phi$ can be derived. The value of cohesion ($C$) is equal to the intercept of the line with the shear strength ($\tau$) axis at $\sigma_n = 0$ and the friction angle ($\phi$) can be obtained by the slope of the shear stress - normal stress curve. The variation of the residual friction angle is from 10° to over 30° from weak rock to hard rock. Generally, with an increase in porosity, moisture, permeability and jointing of the rock sample, the internal friction angle will also decrease. It should be noted that these two parameters are very important and have a great influence on the stability of rock slopes.
3.8 DETERMINATION OF STRENGTH OF ROCK BY THE TRIAXIAL TESTING METHOD

This test evaluates the strength of cylindrical rock specimens. The triaxial compression test has proved to be the most useful test in studying the mechanical properties of rock over a wide range of values for stress and at different temperatures. Triaxial tests on joints can permit large displacements if frictionless end conditions are obtained and corrections are made for changing the joint area; however, the direct shear test is preferable as it permits the control of dilatancy during shearing (Goodman, 1974). Felice et al., (1991) used the X-ray computerised tomography (CT) method for the assessment of test specimen deformation in the post-peak regime. They stated that, by this method, the detection of the transition from homogeneous to nonhomogeneous deformation in a triaxial compression test can be determined. In addition, it can be determined if strain softening is a material property or not. Failure of the rock mass is mostly due to induced shear stresses rather than compressive stress.

The most important shear properties of rock are as follows:

- the triaxial strength ($\sigma_{cf}$) which can be defined as the greatest compressive stress which a specimen can bear in the major principal stress direction when subjected to confining stress $\sigma_3$ or intermediate stresses.

- the cohesion (C) and coefficient of the internal friction angle ($\phi$), these can be obtained from a series of triaxial tests undertaken at different confining pressures.

- the shear strength ($\tau$) can be defined as the shear stress required for failure of the specimen where the normal stress is zero.

In general rock formations are usually in a triaxially stressed state in the field. Therefore, the deformability of rock under such conditions is very important. This is because the bearing capacity of rock, as a natural substructure for a pillar or rock foundation in the design of dams and other structures depends on the mechanism of the deformability of rock. The triaxial compression test is commonly used for the simulation of the stress conditions under which most underground rock masses exist. This test is used to study the shear behaviour of rock.
3.8.1 Preparation of Samples and Testing Procedure

Cylindrical specimens were prepared from rock blocks using a core drilling machine in the laboratory. These specimens were prepared with a diameter of NX size (approximately 54 mm). After preparation of the core samples the ends were cut and ground parallel in accordance with ISRM (1981). Also, the sides of the samples were checked for straightness, smoothness and freedom from irregularities.

3.8.2 Testing procedure

After the core drilling, the cylindrical specimens obtained were used in triaxial testing. The tests were carried out and the vertical load was applied by an automatic hydraulic testing machine of 1800 KN capacity. The confining pressure was applied by using a hand hydraulic pump connected to the chamber of the triaxial cell, and kept constant during the test period. The specimens therefore were confined within a triaxial confining pressure. The chamber was extensive enough to accommodate a specimen with a length to diameter ratio of more than 2. The following procedure was applied to each triaxial compressive test: After replacing the specimen into the triaxial cell, the hydraulic lines were closed and the space around the chamber was filled by the oil. In this condition the confining pressure and the axial load were increased simultaneously until the predetermined values were reached. After that the axial loading continued and the confining pressure was kept constant. During each test the maximum axial load at failure and the corresponding confining pressure ($\sigma_3$) were recorded. It should be noted that after each test the membrane of the triaxial cell must be checked carefully for holes or tears in it. Triaxial tests were carried out on Mugga II, Marulan and Dunmore quarry cylindrical core samples and the results are presented in Figures 3.11 to 3.16 and also in the case study sections of chapters 5, 6, and 7.

3.8.3 Analysis of Triaxial Compression Test Result

Cohesion ($C$) and friction angle ($\phi$) can be calculated from the mean values of the Mohr envelopes. Since the main purpose of this research is the consideration of the strength of
rock and the rock mass, measuring the strength of the rock samples was considered more important than other properties. Presentation of the data as obtained is given in the following forms. Graphs of the triaxial compressive strength ($\sigma_1$) versus the confining pressure ($\sigma_3$) were obtained. The graph of relationship between $\sigma_1$ and $\sigma_3$ is usually curved, but it can be approximated by a straight line over the range under consideration. This line can be represented by the following equation;

$$\sigma_1 = \sigma_c + K \sigma_3 \quad (3.6)$$

where $\sigma_c$ is the uniaxial compressive strength of the rock and $K$ is the slope of the line, $\tan \beta$. In this equation $K$ is known as the triaxial stress factor and may also be shown as

$$K = \tan \beta = \frac{(1 + \sin \phi)}{(1 - \sin \phi)} \quad (3.7)$$

While the value of the uniaxial compressive strength ($\sigma_c$) is almost zero after the rock has broken, the value of $K$ remains unchanged after the movement of the rock by sliding along fracture surfaces. The condition of the rock in the yield zone could, therefore, be represented by;

$$\sigma_1 = K \sigma_3 \quad (3.8)$$

This equation shows that the stress required to cause movement at any point in the yield zone will be $K$ times the confining pressure at that point. The value of $K$ can be obtained from the test carried out on the porphyry rock in the investigation. It may be remembered that this equation will only be valid when sliding occurs along broken surfaces. This condition was dominant in all the tests and all the rocks tested were broken by sliding through a shear plane.
Figure 3.11 Stress - Strain Curve Triaxial Compressive Strength Test (Sections A and B, Porphyry Rock Mugga II Quarry)
Figure 3.12 Stress - Strain Curve Triaxial Compressive Strength Test (Limestone Samples from Marulan Quarry)
Figure 3.13 Results of Triaxial Tests on Intact Rock Samples of Porphyry from Mugga II Quarry in the form of $\sigma_1 - \sigma_3$ Diagram
Figure 3.14 Results of Triaxial Tests on Intact Rock Samples of Weathered Porphyry from Mugga II Quarry in the form of $\sigma_1 - \sigma_3$ Diagram.
Figure 3.15 Results of Triaxial Tests on Intact Rock Samples of Limestone from Marulan Quarry in the form of $\sigma_1 - \sigma_3$ Diagram
Figure 3.16 Results of Triaxial Tests on Intact Rock Samples of Basalt from Dunmore Quarry in the form of $\sigma_1 - \sigma_3$ Diagram

The equation of the line is:

$$y = 12.050x + 131.897 \quad r^2 = 0.940$$
3.9 BRAZILIAN TEST FOR DETERMINATION OF THE TENSILE STRENGTH OF ROCK

3.9.1 Testing Methods

The Brazilian test is an indirect tensile strength test, and the specimens subjected to compression often fail due to the development of tensile stresses. There are two methods used in testing of the tensile strength of rock samples. One is a direct tensile test and the second is an indirect tensile test. Since the direct tensile test is more expensive and presents difficulty in specimen gripping arrangement in comparison with the indirect tensile tests, therefore indirect methods are often used due to their simplicity and cost-effectiveness. The Brazilian test is the most common indirect method for determining rock’s tensile strength. The specimens are disk-shaped with flat and parallel end faces. The aim of this test is to determine the splitting tensile strength of rock by the diametral line compression of a disc.

Once the load is applied to the specimen, a tensile horizontal stress occurs uniformly along the diametral loading direction and a crack starting in this region propagates parallel to the axis of loading. The Brazilian tensile strength, given by \( 0.64 \frac{P}{D^2} \), is approximately equal to the uniaxial tensile strength of the rock (Franklin, 1989). Sun et al (1990) used the Brazilian test for determining the mixed mode I - II fracture. They proposed a cracked Brazilian disk test procedure to determine the envelopes of mixed mode I - II fracture toughness for brittle rocks. Bieniawski, (1967) used Linear Elastic Fracture Mechanics (LEFM) to study rockburst mechanisms and consequently it was found that this tool can be useful for understanding the mechanism of rockbursts in hard rock. The justification for this test is based on the experimental observation that most rocks in a biaxial stress field fail in tension at their uniaxial tensile strength when one principal stress is tensile and the other finite principal stress is compressive, with a magnitude not exceeding three times that of the tensile principal stress (ISRM, 1981). The tensile strength of the specimen \( (\sigma_t) \) shall be calculated by the following equation
\[ \sigma_t = \frac{2P}{\pi D t} \]  

where;
- \( \sigma_t \) is indirect tensile strength (MPa)
- \( P \) is the total load at failure (N)
- \( D \) is the diameter of the test specimen (mm)
- \( t \) is thickness of the specimen (mm)

It should be noted that the tensile strength for each specimen in the sample is expressed as an average. The tensile strength obtained by the Brazilian test is usually greater than that obtained by direct tensile strength testing and, due to this point, is called "Indirect Tensile Strength". The difference is based on the point that, in the Brazilian test, the fracture plane is predetermined along the loading line while in the direct shear test the fracture occurs and propagates along the weakest plane.

The Brazilian test apparatus (Figure 3.17) was used for testing the specimens used in this research. For each rock type, 15 to 20 NX sized samples with a L/D ratio of 0.512 to 0.758 were prepared and tested (Figure 3.18). A constant loading rate of 200 N/s was set up. According to the ISRM's suggestion, a constant rating load of 200 N/s was applied continuously during the test and the different specimens failed within 70 to 320 second.

### 3.9.2 Results of Brazilian Tests and Discussion

Indirect tests for determining the tensile strength of rock are more commonly used than the uniaxial tensile test (direct test). The Brazilian test was used for determination of the tensile strength of the rock specimens. For this purpose, cylindrical samples were prepared. The ends of the specimen were flat to within ±0.025 mm and the surfaces of the cylinder were free from any irregularities according to the ISRM (1981).
Figure 3.17 Illustration of Brazilian test apparatus used for testing the specimens.

Figure 3.18 Representation of some tested porphyry rock samples.
CHAPTER THREE Data Acquisition and Estimation of Rock Mass Strength

Figure 3.17 Illustration of Brazilian test apparatus used for testing the specimens.

Figure 3.18 Representation of some tested porphyry rock samples.
From, between 15 to 20 specimens of different types of rock were used in the tensile strength test and the mean of the results becomes representative of the tensile strength of type of rock. It is believed that for engineering purposes it is probably sufficiently accurate for most problems to assume a tensile strength of 5 - 10% of the uniaxial compressive strength. The results of the measured tensile strengths are shown in Tables 3.6 to 3.9. It should be noted that the tensile strength of a rock specimen can be affected by variations of specimen moisture, specimen geometry, degree of weathering, jointing and also the height to diameter ratio. The tensile strength of rock has great variability and is greatly influenced by specimen size.

3.10 POINT LOAD TEST (INDEX STRENGTH TEST)

Another indirect shear test is the point load tensile test. This test is a simplification of the uniaxial strength test and can give a rapid and accurate strength index, particularly in harder rock, requiring a simple loading frame. The point load tensile strength is determined by applying a compressive point load to the curved surface of a cylindrical core specimen with the axis of the core being horizontal (ISRM, 1985). Figure 3.15 shows the point load test apparatus. The rate of loading is a test variable which affects the compressive strength and the modulus of elasticity. The test is used to evaluate the uniaxial compressive strength of the rock by measuring the point load strength as an index \( I_s \). The test can also measure the strength anisotropy index \( I_a \) which is the ratio of point load strengths in directions which give the greatest and 'at least' values. This test is described by many authors such as, Broch et al, (1972); Bieniawski, (1975); Franklin, (1971), and Hassani, (1980). The point load test offers the benefits of deriving the uniaxial compressive strength in the field with a consequent saving in cost and time, together with the avoidance of sample deterioration. The axial test has been recommended by Broch (1983) for anisotropic rocks, to estimate the amount of strength anisotropy using the anisotropy index

\[
I_a = \frac{I_s \text{ (axial)}}{I_s \text{ (diametral)}} \tag{3.10}
\]

Where;

\( I_s \) = point load strength value (MPa).
Table 3.6 Results of Brazilian test (Specimens from section A & B) Mugga II Quarry

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Length (mm)</th>
<th>Diameter (mm)</th>
<th>L/D Ratio</th>
<th>Load (P) (KN)</th>
<th>Load Rate (N/Sec)</th>
<th>Testing Time (sec)</th>
<th>Tensile Strength (σt (MPa))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>28.6</td>
<td>54.4</td>
<td>0.525</td>
<td>18.5</td>
<td>200</td>
<td>132</td>
<td>7.570</td>
</tr>
<tr>
<td>2</td>
<td>28.0</td>
<td>54.6</td>
<td>0.513</td>
<td>25.0</td>
<td>200</td>
<td>165</td>
<td>10.410</td>
</tr>
<tr>
<td>3</td>
<td>28.5</td>
<td>54.6</td>
<td>0.522</td>
<td>27.5</td>
<td>200</td>
<td>177</td>
<td>11.250</td>
</tr>
<tr>
<td>4</td>
<td>28.6</td>
<td>54.5</td>
<td>0.525</td>
<td>27.0</td>
<td>200</td>
<td>175</td>
<td>11.066</td>
</tr>
<tr>
<td>5</td>
<td>29.0</td>
<td>54.5</td>
<td>0.532</td>
<td>22.5</td>
<td>200</td>
<td>152</td>
<td>9.063</td>
</tr>
<tr>
<td>6</td>
<td>29.0</td>
<td>54.5</td>
<td>0.532</td>
<td>12.5</td>
<td>200</td>
<td>102</td>
<td>5.035</td>
</tr>
<tr>
<td>7</td>
<td>30.0</td>
<td>54.4</td>
<td>0.549</td>
<td>24.5</td>
<td>200</td>
<td>162</td>
<td>9.620</td>
</tr>
<tr>
<td>8</td>
<td>30.0</td>
<td>54.6</td>
<td>0.549</td>
<td>12.2</td>
<td>200</td>
<td>101</td>
<td>4.742</td>
</tr>
<tr>
<td>9</td>
<td>29.1</td>
<td>54.7</td>
<td>0.532</td>
<td>13.6</td>
<td>200</td>
<td>108</td>
<td>5.440</td>
</tr>
<tr>
<td>10</td>
<td>28.0</td>
<td>54.7</td>
<td>0.512</td>
<td>10.0</td>
<td>200</td>
<td>90</td>
<td>4.160</td>
</tr>
<tr>
<td>11</td>
<td>28.5</td>
<td>54.6</td>
<td>0.522</td>
<td>21.6</td>
<td>200</td>
<td>148</td>
<td>8.840</td>
</tr>
<tr>
<td>12</td>
<td>30.0</td>
<td>54.6</td>
<td>0.549</td>
<td>8.9</td>
<td>200</td>
<td>84</td>
<td>3.420</td>
</tr>
<tr>
<td>13</td>
<td>30.0</td>
<td>54.6</td>
<td>0.549</td>
<td>20.8</td>
<td>200</td>
<td>144</td>
<td>8.084</td>
</tr>
<tr>
<td>14</td>
<td>30.0</td>
<td>54.7</td>
<td>0.548</td>
<td>12.5</td>
<td>200</td>
<td>102</td>
<td>4.850</td>
</tr>
<tr>
<td>Average</td>
<td>29.09</td>
<td>54.57</td>
<td>0.532</td>
<td>18.36</td>
<td>200</td>
<td>131.57</td>
<td>7.396</td>
</tr>
</tbody>
</table>
Table 3.7 Results of Brazilian Test (specimens from section C&D ) Mugga II Quarry

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Length (mm)</th>
<th>Diameter (mm)</th>
<th>L/D</th>
<th>Failure Load (KN)</th>
<th>Load Rate (N/Sec)</th>
<th>Test Time (Sec)</th>
<th>Tensile Strength (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>38.50</td>
<td>54.40</td>
<td>0.707</td>
<td>38.50</td>
<td>200</td>
<td>232</td>
<td>11.7</td>
</tr>
<tr>
<td>2</td>
<td>42.00</td>
<td>54.60</td>
<td>0.77</td>
<td>37.00</td>
<td>200</td>
<td>225</td>
<td>10.272</td>
</tr>
<tr>
<td>3</td>
<td>38.50</td>
<td>54.60</td>
<td>0.705</td>
<td>37.60</td>
<td>200</td>
<td>228</td>
<td>11.387</td>
</tr>
<tr>
<td>4</td>
<td>39.10</td>
<td>54.50</td>
<td>0.717</td>
<td>43.60</td>
<td>200</td>
<td>258</td>
<td>13.049</td>
</tr>
<tr>
<td>5</td>
<td>41.40</td>
<td>54.60</td>
<td>0.758</td>
<td>43.60</td>
<td>200</td>
<td>265</td>
<td>12.279</td>
</tr>
<tr>
<td>6</td>
<td>40.00</td>
<td>54.70</td>
<td>0.731</td>
<td>47.60</td>
<td>200</td>
<td>278</td>
<td>13.849</td>
</tr>
<tr>
<td>7</td>
<td>38.00</td>
<td>54.60</td>
<td>0.696</td>
<td>55</td>
<td>200</td>
<td>315</td>
<td>16.875</td>
</tr>
<tr>
<td>8</td>
<td>39.60</td>
<td>54.60</td>
<td>0.725</td>
<td>40.9</td>
<td>200</td>
<td>244</td>
<td>12.042</td>
</tr>
<tr>
<td>9</td>
<td>40.40</td>
<td>54.50</td>
<td>0.741</td>
<td>42</td>
<td>200</td>
<td>250</td>
<td>12.166</td>
</tr>
<tr>
<td>10</td>
<td>39.80</td>
<td>54.50</td>
<td>0.730</td>
<td>39</td>
<td>200</td>
<td>235</td>
<td>11.446</td>
</tr>
<tr>
<td>11</td>
<td>41.20</td>
<td>54.60</td>
<td>0.754</td>
<td>43</td>
<td>200</td>
<td>255</td>
<td>12.169</td>
</tr>
<tr>
<td>12</td>
<td>40.80</td>
<td>54.60</td>
<td>0.747</td>
<td>40.5</td>
<td>200</td>
<td>242</td>
<td>11.573</td>
</tr>
<tr>
<td>13</td>
<td>41.00</td>
<td>54.60</td>
<td>0.750</td>
<td>42.5</td>
<td>200</td>
<td>259</td>
<td>11.80</td>
</tr>
<tr>
<td>14</td>
<td>40.90</td>
<td>54.50</td>
<td>0.750</td>
<td>43.0</td>
<td>200</td>
<td>215</td>
<td>12.280</td>
</tr>
<tr>
<td>Average</td>
<td>40.08</td>
<td>54.56</td>
<td>0.734</td>
<td>42.41</td>
<td>200</td>
<td>250.0</td>
<td>12.348</td>
</tr>
</tbody>
</table>
Table 3.8 Results of Brazilian Test (specimens from Marulan Quarry)

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Length (mm)</th>
<th>Diameter (mm)</th>
<th>L/D</th>
<th>Failure Load (KN)</th>
<th>Loading Rate (N/sec)</th>
<th>Testing Time (sec)</th>
<th>Tensile strength (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>31.50</td>
<td>54.70</td>
<td>0.575</td>
<td>9.60</td>
<td>200</td>
<td>110</td>
<td>3.51</td>
</tr>
<tr>
<td>2</td>
<td>31.60</td>
<td>54.70</td>
<td>0.577</td>
<td>8.50</td>
<td>200</td>
<td>95</td>
<td>3.13</td>
</tr>
<tr>
<td>3</td>
<td>33.00</td>
<td>54.70</td>
<td>0.603</td>
<td>8.40</td>
<td>200</td>
<td>90</td>
<td>2.96</td>
</tr>
<tr>
<td>4</td>
<td>34.00</td>
<td>54.70</td>
<td>0.621</td>
<td>10.20</td>
<td>200</td>
<td>125</td>
<td>3.49</td>
</tr>
<tr>
<td>5</td>
<td>33.90</td>
<td>54.70</td>
<td>0.619</td>
<td>7.50</td>
<td>200</td>
<td>80</td>
<td>2.57</td>
</tr>
<tr>
<td>6</td>
<td>33.00</td>
<td>54.70</td>
<td>0.603</td>
<td>7.80</td>
<td>200</td>
<td>75</td>
<td>2.74</td>
</tr>
<tr>
<td>7</td>
<td>33.00</td>
<td>54.70</td>
<td>0.603</td>
<td>7.90</td>
<td>200</td>
<td>70</td>
<td>2.80</td>
</tr>
<tr>
<td>8</td>
<td>32.20</td>
<td>54.70</td>
<td>0.588</td>
<td>12.40</td>
<td>200</td>
<td>90</td>
<td>4.48</td>
</tr>
<tr>
<td>9</td>
<td>34.00</td>
<td>54.60</td>
<td>0.622</td>
<td>7.80</td>
<td>200</td>
<td>85</td>
<td>2.67</td>
</tr>
<tr>
<td>10</td>
<td>32.50</td>
<td>54.60</td>
<td>0.595</td>
<td>9.00</td>
<td>200</td>
<td>60</td>
<td>3.22</td>
</tr>
<tr>
<td>11</td>
<td>32.60</td>
<td>54.70</td>
<td>0.596</td>
<td>8.20</td>
<td>200</td>
<td>80</td>
<td>2.93</td>
</tr>
<tr>
<td>12</td>
<td>33.20</td>
<td>54.70</td>
<td>0.607</td>
<td>12.40</td>
<td>200</td>
<td>150</td>
<td>4.35</td>
</tr>
<tr>
<td>13</td>
<td>31.50</td>
<td>54.60</td>
<td>0.576</td>
<td>16.10</td>
<td>200</td>
<td>240</td>
<td>5.41</td>
</tr>
<tr>
<td>14</td>
<td>30.30</td>
<td>54.60</td>
<td>0.558</td>
<td>9.80</td>
<td>200</td>
<td>140</td>
<td>3.74</td>
</tr>
<tr>
<td>15</td>
<td>33.00</td>
<td>54.70</td>
<td>0.603</td>
<td>15.60</td>
<td>200</td>
<td>235</td>
<td>5.49</td>
</tr>
<tr>
<td>16</td>
<td>33.90</td>
<td>54.60</td>
<td>0.620</td>
<td>14.50</td>
<td>200</td>
<td>145</td>
<td>4.97</td>
</tr>
<tr>
<td>17</td>
<td>32.00</td>
<td>54.70</td>
<td>0.585</td>
<td>16.20</td>
<td>200</td>
<td>240</td>
<td>5.88</td>
</tr>
<tr>
<td>18</td>
<td>33.50</td>
<td>54.70</td>
<td>0.612</td>
<td>12.20</td>
<td>200</td>
<td>90</td>
<td>4.23</td>
</tr>
<tr>
<td>19</td>
<td>33.00</td>
<td>54.70</td>
<td>0.603</td>
<td>12.60</td>
<td>200</td>
<td>140</td>
<td>4.44</td>
</tr>
<tr>
<td>20</td>
<td>32.80</td>
<td>54.70</td>
<td>0.599</td>
<td>9.20</td>
<td>200</td>
<td>120</td>
<td>3.26</td>
</tr>
<tr>
<td>Average</td>
<td>32.725</td>
<td>54.675</td>
<td>0.597</td>
<td>10.795</td>
<td>200</td>
<td>124</td>
<td>3.813</td>
</tr>
</tbody>
</table>
Table 3.9  Results of Brazilian Test (specimens from Dunmore Quarry)

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Length (mm)</th>
<th>Diameter (mm)</th>
<th>L/D Ratio</th>
<th>Failure Load (KN)</th>
<th>Loading Rate (N/sec)</th>
<th>Testing Time (sec)</th>
<th>Tensile strength (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>31.30</td>
<td>54.70</td>
<td>0.572</td>
<td>25.60</td>
<td>200</td>
<td>280</td>
<td>9.51</td>
</tr>
<tr>
<td>2</td>
<td>31.40</td>
<td>54.70</td>
<td>0.574</td>
<td>19.30</td>
<td>200</td>
<td>250</td>
<td>7.15</td>
</tr>
<tr>
<td>3</td>
<td>31.50</td>
<td>54.70</td>
<td>0.575</td>
<td>27.50</td>
<td>200</td>
<td>300</td>
<td>10.15</td>
</tr>
<tr>
<td>4</td>
<td>31.80</td>
<td>54.70</td>
<td>0.581</td>
<td>25.30</td>
<td>200</td>
<td>220</td>
<td>9.25</td>
</tr>
<tr>
<td>5</td>
<td>31.80</td>
<td>54.70</td>
<td>0.581</td>
<td>24.60</td>
<td>200</td>
<td>225</td>
<td>8.99</td>
</tr>
<tr>
<td>6</td>
<td>30.30</td>
<td>54.70</td>
<td>0.554</td>
<td>27.80</td>
<td>200</td>
<td>270</td>
<td>10.67</td>
</tr>
<tr>
<td>7</td>
<td>32.00</td>
<td>54.70</td>
<td>0.585</td>
<td>32.80</td>
<td>200</td>
<td>280</td>
<td>11.92</td>
</tr>
<tr>
<td>8</td>
<td>31.70</td>
<td>54.70</td>
<td>0.579</td>
<td>36.70</td>
<td>200</td>
<td>320</td>
<td>13.46</td>
</tr>
<tr>
<td>9</td>
<td>31.80</td>
<td>54.70</td>
<td>0.581</td>
<td>32.10</td>
<td>200</td>
<td>220</td>
<td>11.74</td>
</tr>
<tr>
<td>10</td>
<td>30.70</td>
<td>54.70</td>
<td>0.561</td>
<td>21.50</td>
<td>200</td>
<td>200</td>
<td>8.14</td>
</tr>
<tr>
<td>11</td>
<td>33.00</td>
<td>54.70</td>
<td>0.603</td>
<td>23.60</td>
<td>200</td>
<td>180</td>
<td>8.31</td>
</tr>
<tr>
<td>12</td>
<td>31.80</td>
<td>54.70</td>
<td>0.581</td>
<td>23.90</td>
<td>200</td>
<td>170</td>
<td>8.74</td>
</tr>
<tr>
<td>13</td>
<td>32.40</td>
<td>54.70</td>
<td>0.592</td>
<td>22.20</td>
<td>200</td>
<td>150</td>
<td>7.97</td>
</tr>
<tr>
<td>14</td>
<td>30.40</td>
<td>54.60</td>
<td>0.556</td>
<td>25.20</td>
<td>200</td>
<td>180</td>
<td>9.66</td>
</tr>
<tr>
<td>15</td>
<td>33.00</td>
<td>54.70</td>
<td>0.603</td>
<td>26.10</td>
<td>200</td>
<td>155</td>
<td>9.19</td>
</tr>
<tr>
<td>16</td>
<td>31.30</td>
<td>54.60</td>
<td>0.573</td>
<td>24.80</td>
<td>200</td>
<td>140</td>
<td>9.23</td>
</tr>
<tr>
<td>17</td>
<td>32.40</td>
<td>54.70</td>
<td>0.592</td>
<td>25.80</td>
<td>200</td>
<td>180</td>
<td>9.26</td>
</tr>
<tr>
<td>18</td>
<td>33.30</td>
<td>54.70</td>
<td>0.608</td>
<td>24.70</td>
<td>200</td>
<td>150</td>
<td>8.62</td>
</tr>
<tr>
<td>19</td>
<td>31.50</td>
<td>54.70</td>
<td>0.575</td>
<td>20.50</td>
<td>200</td>
<td>140</td>
<td>7.57</td>
</tr>
<tr>
<td>20</td>
<td>31.30</td>
<td>54.60</td>
<td>0.573</td>
<td>21.80</td>
<td>200</td>
<td>140</td>
<td>8.11</td>
</tr>
<tr>
<td>21</td>
<td>30.00</td>
<td>54.70</td>
<td>0.548</td>
<td>24.20</td>
<td>200</td>
<td>165</td>
<td>9.38</td>
</tr>
<tr>
<td>22</td>
<td>30.10</td>
<td>54.60</td>
<td>0.551</td>
<td>20.50</td>
<td>200</td>
<td>135</td>
<td>8.32</td>
</tr>
<tr>
<td>23</td>
<td>33.00</td>
<td>54.70</td>
<td>0.603</td>
<td>31.5</td>
<td>200</td>
<td>200</td>
<td>11.09</td>
</tr>
<tr>
<td>Average</td>
<td>31.647</td>
<td>54.68</td>
<td>0.578</td>
<td>25.59</td>
<td>200</td>
<td>202.17</td>
<td>9.42</td>
</tr>
</tbody>
</table>
The results of investigation by Bieniawski, (1975) and Hassani et al, (1980) have shown that more accurate and repeatable index values are derived from diametral testing. In the case of anisotropic rocks, the direction of preparation of samples is very important, because in the direction parallel to the planes of anisotropy, rock shows low strength values and in the direction of the normal to the planes of anisotropy, rock shows the greatest strength values. In other words, for relatively homogeneous rock, the diametral point load test is a reliable method of indexing rock strength.

The point load test apparatus (Figure 3.19) consists of a loading system comprising a load frame, hydraulic pump, ram, and platens. A gauge system for measuring the load \( P \) is needed for breaking the specimen, together with a system for measuring the distance \( D \) between two platen contact points. In this apparatus the loading frame is designed to accept specimens up to 100 mm in diameter, so that it could be used for most common sizes of rock core samples.

### 3.10.1 Testing Methods

There are four different methods for this test. The diametral and axial tests with saw cut faces are the most accurate and were selected for the purpose of this investigation. Since rocks are not homogenous and show anisotropy in their mechanical properties, it would be misleading to classify the strength of rock material by a single number which bears no relation to direction. In the diametral point load test, the failure load \( (P) \) is independent of the length of the specimen where the distance \( L \) is sufficiently large. In this test core samples with a length to diameter ratio of around 1.5 were used. According to the ISRM (1981) in the case of lateral point load test the length of the core specimens must be more than \( \frac{1}{2} D \), in other words it must be \( L > .05 D \) but in the case of axial point load test the load is applied to the axis of the core samples and the length to diameter ratio is between \( 0.3 D \) to \( D \).

However, after preparation of the samples they were used for diametral and axial tests (Figure 3.20 ). For this purpose the core samples were inserted into the apparatus and cones were closed together to make contact with the core sample. The distance \( D \) was then recorded and load was continuously increased so that the failure occurred within 15 to 70 seconds.
Figure 3.19 Illustration of point load test apparatus used for testing.

Figure 3.20 Some of the samples tested by point load test apparatus.
The failure load $P$ was then recorded from the gauge. In this research both axial and diametral specimens were used. The length to diameter ratio in the axial test was 1, and in the case of diametral test the length to diameter ratio was 1.5. It should be noted that the test is valid when a clean diametral failure occurs between two parts of the core samples. The specimens will fail at a low applied force ($P$) due to the tensile stresses over the diametral area between the cones. The point load index ($I_s$) shows the strength of the specimens at failure:

$$I_s = \frac{P}{D^2} \tag{3.11}$$

The point load index, according to the experimental works, has a very close correlation with the uniaxial compressive strength. This correlation is shown by the following equation.

$$\sigma_c = K I_s \tag{3.12}$$

Where;

$K$ is a constant value.

Brook (1980), proposed the following relationship between compressive strength and point load index ($T_{500}$).

$$C_0 = 12.5 T_{500} \tag{3.13}$$

where;

the strength index ($T_{500}$) is the load for a cross-section area of 500 mm$^2$, divided by this area; i.e.

$$T_{500} = \frac{\text{load at 500 (mm)$^2$}}{500 \text{ (mm)$^2$}} \tag{3.14}$$

The size correction of the point load test to a chosen standard size is possible using the expression
\[ T_{500} = 2115 \frac{P}{A^{0.75}} \text{ (Mpa)} \]  

\( P \) = load (kN)  
\( A \) = area (mm)\(^2\)

### 3.10.2 Results of Point Load Test and Discussion

The point load index is used to estimate the uniaxial compressive strength and tensile strength of the rock samples as expressed in Equation (3.12). From each types of rock, between 14 to 24 samples were tested and the results show very close relationship to each other except in a few special cases where the samples were containing joints or weakness planes. Testing results and the specification of samples are presented in Tables 3.10 to 3.17. A correlation between axial and diametral methods shows a very high corresponding correlation coefficient of \( r^2 = 0.997 \) and \( r^2 = 0.983 \) in power and linear regression respectively. From the results it was concluded that for most hard and brittle rocks, the ratio of compressive to tensile strength is approx. 10 to 15. The consistency of the test is suitable for hard rocks and it is recommended for measuring the indirect tensile and compressive strength of the hard rock samples.

### 3.11 DETERMINATION OF STRENGTH OF ROCK BY SCHMIDT HAMMER

There are different methods for considering the joint compressive strength (JCS). In this research a N-type Schmidt hammer apparatus were used for measurement of the Joint compressive strength of jointed rocks. For each type of rock, between 150 to 200 reading were carried out on the slope faces of the different types of quarries. The results and discussions are expressed in detail in chapter 4. For more information, refer to section 4.5.3.
Table 3.10  Results of Point load tests (axial) from sections A & B based on the equation: $I_s (50) = F \cdot P / D e^2$

<table>
<thead>
<tr>
<th>Number</th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>(L/D)</th>
<th>P (KN)</th>
<th>$D e^2$</th>
<th>F</th>
<th>$I_s (50)$</th>
<th>$\sigma_e$</th>
<th>$\sigma_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>54.40</td>
<td>28.00</td>
<td>0.51</td>
<td>8.01</td>
<td>1940.38</td>
<td>0.94</td>
<td>2.56</td>
<td>61.38</td>
<td>3.20</td>
</tr>
<tr>
<td>2</td>
<td>54.50</td>
<td>28.00</td>
<td>0.51</td>
<td>2.68</td>
<td>1943.95</td>
<td>0.94</td>
<td>0.85</td>
<td>20.46</td>
<td>1.07</td>
</tr>
<tr>
<td>3</td>
<td>54.40</td>
<td>28.00</td>
<td>0.51</td>
<td>10.68</td>
<td>1940.38</td>
<td>0.94</td>
<td>3.41</td>
<td>81.84</td>
<td>4.26</td>
</tr>
<tr>
<td>4</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>10.68</td>
<td>1947.52</td>
<td>0.94</td>
<td>3.41</td>
<td>81.84</td>
<td>4.26</td>
</tr>
<tr>
<td>5</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>6.68</td>
<td>1947.52</td>
<td>0.94</td>
<td>2.13</td>
<td>51.15</td>
<td>2.66</td>
</tr>
<tr>
<td>6</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>3.35</td>
<td>1947.52</td>
<td>0.94</td>
<td>1.07</td>
<td>25.68</td>
<td>1.34</td>
</tr>
<tr>
<td>7</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>4.67</td>
<td>1947.52</td>
<td>0.94</td>
<td>1.49</td>
<td>35.80</td>
<td>1.86</td>
</tr>
<tr>
<td>8</td>
<td>54.40</td>
<td>28.00</td>
<td>0.51</td>
<td>4.67</td>
<td>1940.38</td>
<td>0.94</td>
<td>0.85</td>
<td>35.80</td>
<td>1.86</td>
</tr>
<tr>
<td>9</td>
<td>54.50</td>
<td>28.00</td>
<td>0.51</td>
<td>2.67</td>
<td>1943.95</td>
<td>0.94</td>
<td>3.84</td>
<td>20.46</td>
<td>1.07</td>
</tr>
<tr>
<td>10</td>
<td>54.50</td>
<td>28.00</td>
<td>0.51</td>
<td>12.02</td>
<td>1943.95</td>
<td>0.94</td>
<td>3.41</td>
<td>92.07</td>
<td>4.80</td>
</tr>
<tr>
<td>11</td>
<td>54.50</td>
<td>28.00</td>
<td>0.51</td>
<td>10.68</td>
<td>1943.95</td>
<td>0.94</td>
<td>3.41</td>
<td>81.84</td>
<td>4.26</td>
</tr>
<tr>
<td>12</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>10.68</td>
<td>1947.52</td>
<td>0.94</td>
<td>3.41</td>
<td>81.84</td>
<td>4.26</td>
</tr>
<tr>
<td>13</td>
<td>54.60</td>
<td>29.00</td>
<td>0.53</td>
<td>10.68</td>
<td>2017.07</td>
<td>0.94</td>
<td>3.84</td>
<td>81.84</td>
<td>4.26</td>
</tr>
<tr>
<td>14</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>12.02</td>
<td>1947.52</td>
<td>0.94</td>
<td>2.79</td>
<td>92.07</td>
<td>4.80</td>
</tr>
<tr>
<td><strong>Mean value</strong></td>
<td>54.53</td>
<td>28.07</td>
<td>0.51</td>
<td>8.73</td>
<td>19.46.58</td>
<td>0.94</td>
<td>2.79</td>
<td>66.92</td>
<td>3.48</td>
</tr>
</tbody>
</table>
Table 3.11 Results of Point load test (diametral) from sections A&B based on the equation: $I_s(50) = \frac{F \cdot P}{D^2}$

<table>
<thead>
<tr>
<th>Number</th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>(L/D)</th>
<th>P (KN)</th>
<th>$D^2$</th>
<th>F</th>
<th>$I_s (50)$</th>
<th>$\sigma_c$</th>
<th>$\sigma_t$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>54.4</td>
<td>60</td>
<td>1.102</td>
<td>13.355</td>
<td>2959.36</td>
<td>1.0386</td>
<td>4.71</td>
<td>112.97</td>
<td>5.88</td>
</tr>
<tr>
<td>2</td>
<td>54.5</td>
<td>59</td>
<td>1.082</td>
<td>9.348</td>
<td>2970.25</td>
<td>1.039</td>
<td>3.29</td>
<td>78.86</td>
<td>4.11</td>
</tr>
<tr>
<td>3</td>
<td>54.4</td>
<td>59</td>
<td>1.084</td>
<td>9.348</td>
<td>2959.36</td>
<td>1.038</td>
<td>3.29</td>
<td>79.08</td>
<td>4.12</td>
</tr>
<tr>
<td>4</td>
<td>54.6</td>
<td>60</td>
<td>1.098</td>
<td>14.69</td>
<td>2981.16</td>
<td>1.04</td>
<td>5.15</td>
<td>123.58</td>
<td>6.44</td>
</tr>
<tr>
<td>5</td>
<td>54.6</td>
<td>60</td>
<td>1.098</td>
<td>13.355</td>
<td>2981.16</td>
<td>1.04</td>
<td>4.68</td>
<td>112.35</td>
<td>5.85</td>
</tr>
<tr>
<td>6</td>
<td>54.6</td>
<td>62</td>
<td>1.135</td>
<td>14.69</td>
<td>2981.16</td>
<td>1.04</td>
<td>5.15</td>
<td>123.58</td>
<td>6.44</td>
</tr>
<tr>
<td>7</td>
<td>54.6</td>
<td>60</td>
<td>1.098</td>
<td>8.013</td>
<td>2981.16</td>
<td>1.04</td>
<td>2.81</td>
<td>67.41</td>
<td>3.51</td>
</tr>
<tr>
<td>8</td>
<td>54.4</td>
<td>60</td>
<td>1.102</td>
<td>12.019</td>
<td>2959.36</td>
<td>1.0386</td>
<td>4.24</td>
<td>101.67</td>
<td>5.30</td>
</tr>
<tr>
<td>9</td>
<td>54.5</td>
<td>59</td>
<td>1.084</td>
<td>10.684</td>
<td>2970.25</td>
<td>1.0386</td>
<td>3.76</td>
<td>90.13</td>
<td>4.69</td>
</tr>
<tr>
<td>10</td>
<td>54.5</td>
<td>58</td>
<td>1.094</td>
<td>13.355</td>
<td>2970.25</td>
<td>1.039</td>
<td>4.69</td>
<td>112.66</td>
<td>5.87</td>
</tr>
<tr>
<td>11</td>
<td>54.5</td>
<td>60</td>
<td>1.098</td>
<td>13.355</td>
<td>2981.16</td>
<td>1.04</td>
<td>4.68</td>
<td>112.25</td>
<td>5.85</td>
</tr>
<tr>
<td>12</td>
<td>54.6</td>
<td>60</td>
<td>1.098</td>
<td>6.677</td>
<td>2981.16</td>
<td>1.04</td>
<td>2.34</td>
<td>56.17</td>
<td>2.93</td>
</tr>
<tr>
<td>13</td>
<td>54.6</td>
<td>60</td>
<td>1.098</td>
<td>10.684</td>
<td>2981.16</td>
<td>1.04</td>
<td>3.75</td>
<td>89.88</td>
<td>4.68</td>
</tr>
<tr>
<td>14</td>
<td>54.6</td>
<td>60</td>
<td>1.098</td>
<td>13.355</td>
<td>2981.16</td>
<td>1.04</td>
<td>4.68</td>
<td>112.35</td>
<td>5.85</td>
</tr>
<tr>
<td>Mean Value</td>
<td>54.53</td>
<td>59.78</td>
<td>1.09</td>
<td>11.64</td>
<td>2974.15</td>
<td>1.04</td>
<td>4.05</td>
<td>98.07</td>
<td>5.11</td>
</tr>
</tbody>
</table>
Table 3.12  Point load test (axial) from section C & D based on the equation: Is (50) = F.P/De^2

<table>
<thead>
<tr>
<th></th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>(L/D)</th>
<th>P (KN)</th>
<th>De2</th>
<th>F</th>
<th>1s (50)</th>
<th>σh</th>
<th>σv</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>54.40</td>
<td>28.00</td>
<td>0.51</td>
<td>26.05</td>
<td>1940.38</td>
<td>0.94</td>
<td>12.68</td>
<td>304.29</td>
<td>15.85</td>
</tr>
<tr>
<td>2</td>
<td>54.50</td>
<td>28.00</td>
<td>0.51</td>
<td>6.68</td>
<td>1943.95</td>
<td>0.94</td>
<td>3.24</td>
<td>77.87</td>
<td>4.06</td>
</tr>
<tr>
<td>3</td>
<td>54.40</td>
<td>28.00</td>
<td>0.51</td>
<td>25.38</td>
<td>1940.38</td>
<td>0.94</td>
<td>12.35</td>
<td>296.46</td>
<td>15.44</td>
</tr>
<tr>
<td>4</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>21.37</td>
<td>1947.52</td>
<td>0.94</td>
<td>10.36</td>
<td>248.73</td>
<td>12.95</td>
</tr>
<tr>
<td>5</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>21.37</td>
<td>1947.52</td>
<td>0.94</td>
<td>10.36</td>
<td>248.73</td>
<td>12.95</td>
</tr>
<tr>
<td>6</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>22.70</td>
<td>1947.52</td>
<td>0.94</td>
<td>11.01</td>
<td>264.27</td>
<td>13.76</td>
</tr>
<tr>
<td>7</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>20.03</td>
<td>1947.52</td>
<td>0.94</td>
<td>9.72</td>
<td>233.18</td>
<td>12.14</td>
</tr>
<tr>
<td>8</td>
<td>54.40</td>
<td>28.00</td>
<td>0.51</td>
<td>28.05</td>
<td>1940.38</td>
<td>0.94</td>
<td>13.65</td>
<td>327.66</td>
<td>17.07</td>
</tr>
<tr>
<td>9</td>
<td>54.50</td>
<td>28.00</td>
<td>0.51</td>
<td>22.70</td>
<td>1943.95</td>
<td>0.94</td>
<td>11.03</td>
<td>264.76</td>
<td>13.79</td>
</tr>
<tr>
<td>10</td>
<td>54.50</td>
<td>28.00</td>
<td>0.51</td>
<td>24.04</td>
<td>1943.95</td>
<td>0.94</td>
<td>11.68</td>
<td>280.34</td>
<td>14.60</td>
</tr>
<tr>
<td>11</td>
<td>54.50</td>
<td>28.00</td>
<td>0.51</td>
<td>22.70</td>
<td>1943.95</td>
<td>0.94</td>
<td>11.03</td>
<td>264.76</td>
<td>13.79</td>
</tr>
<tr>
<td>12</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>21.37</td>
<td>1947.52</td>
<td>0.94</td>
<td>10.36</td>
<td>248.72</td>
<td>12.95</td>
</tr>
<tr>
<td>13</td>
<td>54.60</td>
<td>29.00</td>
<td>0.53</td>
<td>24.04</td>
<td>2017.07</td>
<td>0.94</td>
<td>11.26</td>
<td>270.18</td>
<td>14.07</td>
</tr>
<tr>
<td>14</td>
<td>54.60</td>
<td>28.00</td>
<td>0.51</td>
<td>21.37</td>
<td>1947.52</td>
<td>0.94</td>
<td>10.36</td>
<td>248.73</td>
<td>12.95</td>
</tr>
<tr>
<td>Mean value</td>
<td>54.53</td>
<td>28.00</td>
<td>0.511</td>
<td>23.17</td>
<td>1949.94</td>
<td>0.94</td>
<td>11.22</td>
<td>269.29</td>
<td>14.02</td>
</tr>
</tbody>
</table>
Table 3.13 Results of Point load tests (diametral) from sections C & D based on the equation: $I_{s(50)} = \frac{F.P}{D^2}$

<table>
<thead>
<tr>
<th>Number</th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>(L/D)</th>
<th>P (KN)</th>
<th>$D^2$</th>
<th>F</th>
<th>Is (50)</th>
<th>$\sigma_e$</th>
<th>$\sigma_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>54.4</td>
<td>65</td>
<td>1.19</td>
<td>26.71</td>
<td>2959.36</td>
<td>1.04</td>
<td>9.41</td>
<td>225.94</td>
<td>11.77</td>
</tr>
<tr>
<td>2</td>
<td>54.5</td>
<td>66</td>
<td>1.21</td>
<td>21.368</td>
<td>2970.25</td>
<td>1.04</td>
<td>7.50</td>
<td>180.09</td>
<td>9.38</td>
</tr>
<tr>
<td>3</td>
<td>54.4</td>
<td>64.5</td>
<td>1.19</td>
<td>26.71</td>
<td>2959.36</td>
<td>1.04</td>
<td>9.41</td>
<td>225.94</td>
<td>11.77</td>
</tr>
<tr>
<td>4</td>
<td>54.6</td>
<td>64.5</td>
<td>1.18</td>
<td>8.013</td>
<td>2981.16</td>
<td>1.04</td>
<td>2.80</td>
<td>67.29</td>
<td>3.50</td>
</tr>
<tr>
<td>5</td>
<td>54.6</td>
<td>64.5</td>
<td>1.18</td>
<td>26.71</td>
<td>2981.16</td>
<td>1.04</td>
<td>9.35</td>
<td>224.29</td>
<td>11.68</td>
</tr>
<tr>
<td>6</td>
<td>54.6</td>
<td>64.5</td>
<td>1.18</td>
<td>13.355</td>
<td>2981.16</td>
<td>1.04</td>
<td>4.67</td>
<td>112.15</td>
<td>5.84</td>
</tr>
<tr>
<td>7</td>
<td>54.6</td>
<td>64.5</td>
<td>1.18</td>
<td>20.032</td>
<td>2981.16</td>
<td>1.04</td>
<td>7.01</td>
<td>168.21</td>
<td>8.76</td>
</tr>
<tr>
<td>8</td>
<td>54.4</td>
<td>64.5</td>
<td>1.19</td>
<td>28.045</td>
<td>2959.36</td>
<td>1.04</td>
<td>9.88</td>
<td>237.24</td>
<td>12.36</td>
</tr>
<tr>
<td>9</td>
<td>54.5</td>
<td>65</td>
<td>1.19</td>
<td>13.355</td>
<td>2970.25</td>
<td>1.04</td>
<td>4.69</td>
<td>112.56</td>
<td>5.86</td>
</tr>
<tr>
<td>10</td>
<td>54.5</td>
<td>64</td>
<td>1.17</td>
<td>29.381</td>
<td>2970.25</td>
<td>1.04</td>
<td>10.32</td>
<td>247.63</td>
<td>12.90</td>
</tr>
<tr>
<td>11</td>
<td>54.5</td>
<td>65</td>
<td>1.19</td>
<td>24.039</td>
<td>2981.16</td>
<td>1.04</td>
<td>8.41</td>
<td>201.86</td>
<td>10.51</td>
</tr>
<tr>
<td>12</td>
<td>54.6</td>
<td>64.5</td>
<td>1.18</td>
<td>26.71</td>
<td>2981.16</td>
<td>1.04</td>
<td>9.35</td>
<td>224.29</td>
<td>11.68</td>
</tr>
<tr>
<td>13</td>
<td>54.6</td>
<td>65</td>
<td>1.19</td>
<td>25.377</td>
<td>2981.16</td>
<td>1.04</td>
<td>8.88</td>
<td>213.10</td>
<td>11.10</td>
</tr>
<tr>
<td>14</td>
<td>54.6</td>
<td>64.5</td>
<td>1.18</td>
<td>20.032</td>
<td>2981.16</td>
<td>1.04</td>
<td>7.01</td>
<td>168.21</td>
<td>8.76</td>
</tr>
<tr>
<td>Mean Value</td>
<td>54.53</td>
<td>59.78</td>
<td>1.09</td>
<td>11.64</td>
<td>2974.15</td>
<td>1.04</td>
<td>4.08</td>
<td>98.07</td>
<td>5.11</td>
</tr>
</tbody>
</table>
Table 3.14 Results of Point load tests (specimens from Marulan quarry) based on the equation \( I_s (50) = \frac{F.P}{De^2} \)

<table>
<thead>
<tr>
<th>Number</th>
<th>Type</th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>(L/D)</th>
<th>P (KN)</th>
<th>De^2</th>
<th>F</th>
<th>( I_s (50) )</th>
<th>( \sigma_c^* )</th>
<th>( \sigma_t^* )</th>
<th>( \sigma_c^{**} )</th>
<th>( \sigma_t^{**} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>54.70</td>
<td>41.70</td>
<td>0.76</td>
<td>10.01</td>
<td>2905.72</td>
<td>1.03</td>
<td>3.50</td>
<td>83.97</td>
<td>4.37</td>
<td>101.47</td>
<td>10.50</td>
</tr>
<tr>
<td>2</td>
<td>A</td>
<td>54.70</td>
<td>42.00</td>
<td>0.77</td>
<td>10.14</td>
<td>2926.62</td>
<td>1.03</td>
<td>3.55</td>
<td>85.20</td>
<td>4.44</td>
<td>102.95</td>
<td>10.65</td>
</tr>
<tr>
<td>3</td>
<td>A</td>
<td>54.70</td>
<td>41.80</td>
<td>0.76</td>
<td>12.01</td>
<td>2912.69</td>
<td>1.03</td>
<td>4.20</td>
<td>100.81</td>
<td>5.25</td>
<td>121.81</td>
<td>12.60</td>
</tr>
<tr>
<td>4</td>
<td>A</td>
<td>54.70</td>
<td>42.00</td>
<td>0.77</td>
<td>10.68</td>
<td>2926.62</td>
<td>1.04</td>
<td>3.74</td>
<td>89.74</td>
<td>4.67</td>
<td>108.43</td>
<td>11.22</td>
</tr>
<tr>
<td>5</td>
<td>A</td>
<td>54.70</td>
<td>41.60</td>
<td>0.76</td>
<td>6.68</td>
<td>2947.53</td>
<td>1.03</td>
<td>4.92</td>
<td>117.99</td>
<td>5.99</td>
<td>139.02</td>
<td>14.38</td>
</tr>
<tr>
<td>6</td>
<td>A</td>
<td>54.70</td>
<td>41.70</td>
<td>0.76</td>
<td>8.28</td>
<td>2905.72</td>
<td>1.04</td>
<td>2.90</td>
<td>69.57</td>
<td>3.62</td>
<td>84.07</td>
<td>8.70</td>
</tr>
<tr>
<td>7</td>
<td>A</td>
<td>54.70</td>
<td>42.00</td>
<td>0.77</td>
<td>8.01</td>
<td>2947.53</td>
<td>1.04</td>
<td>2.34</td>
<td>56.22</td>
<td>2.93</td>
<td>67.93</td>
<td>7.03</td>
</tr>
<tr>
<td>8</td>
<td>A</td>
<td>54.70</td>
<td>42.00</td>
<td>0.77</td>
<td>8.34</td>
<td>2898.75</td>
<td>1.04</td>
<td>2.80</td>
<td>67.20</td>
<td>3.50</td>
<td>81.19</td>
<td>8.40</td>
</tr>
<tr>
<td>9</td>
<td>A</td>
<td>54.70</td>
<td>42.30</td>
<td>0.77</td>
<td>6.68</td>
<td>2926.62</td>
<td>1.03</td>
<td>4.20</td>
<td>90.22</td>
<td>4.70</td>
<td>109.01</td>
<td>11.28</td>
</tr>
<tr>
<td>10</td>
<td>A</td>
<td>54.70</td>
<td>42.70</td>
<td>0.78</td>
<td>6.68</td>
<td>2926.62</td>
<td>1.04</td>
<td>3.52</td>
<td>84.47</td>
<td>4.40</td>
<td>102.06</td>
<td>10.56</td>
</tr>
<tr>
<td>11</td>
<td>A</td>
<td>54.70</td>
<td>42.50</td>
<td>0.78</td>
<td>13.36</td>
<td>2961.46</td>
<td>1.04</td>
<td>4.69</td>
<td>112.56</td>
<td>5.86</td>
<td>136.01</td>
<td>14.07</td>
</tr>
<tr>
<td>12</td>
<td>A</td>
<td>54.70</td>
<td>43.00</td>
<td>0.79</td>
<td>14.02</td>
<td>2947.53</td>
<td>1.04</td>
<td>4.92</td>
<td>117.99</td>
<td>6.15</td>
<td>142.57</td>
<td>14.75</td>
</tr>
<tr>
<td>13</td>
<td>A</td>
<td>54.70</td>
<td>43.00</td>
<td>0.79</td>
<td>13.62</td>
<td>2996.31</td>
<td>1.04</td>
<td>4.79</td>
<td>115.05</td>
<td>5.99</td>
<td>139.02</td>
<td>14.38</td>
</tr>
<tr>
<td>14</td>
<td>A</td>
<td>54.70</td>
<td>42.70</td>
<td>0.78</td>
<td>13.89</td>
<td>2975.40</td>
<td>1.04</td>
<td>4.88</td>
<td>117.15</td>
<td>6.10</td>
<td>141.55</td>
<td>14.64</td>
</tr>
<tr>
<td>15</td>
<td>A</td>
<td>54.70</td>
<td>44.80</td>
<td>0.82</td>
<td>9.34</td>
<td>3121.73</td>
<td>1.05</td>
<td>3.32</td>
<td>79.63</td>
<td>4.15</td>
<td>96.22</td>
<td>9.95</td>
</tr>
<tr>
<td>16</td>
<td>A</td>
<td>54.70</td>
<td>43.00</td>
<td>0.79</td>
<td>10.01</td>
<td>2996.31</td>
<td>1.04</td>
<td>3.52</td>
<td>84.56</td>
<td>4.40</td>
<td>102.17</td>
<td>10.57</td>
</tr>
<tr>
<td>17</td>
<td>A</td>
<td>54.70</td>
<td>43.10</td>
<td>0.79</td>
<td>13.35</td>
<td>3003.27</td>
<td>1.04</td>
<td>4.70</td>
<td>112.83</td>
<td>5.88</td>
<td>136.33</td>
<td>14.10</td>
</tr>
<tr>
<td>18</td>
<td>A</td>
<td>54.70</td>
<td>43.00</td>
<td>0.79</td>
<td>10.68</td>
<td>2996.31</td>
<td>1.04</td>
<td>3.76</td>
<td>90.22</td>
<td>4.70</td>
<td>109.01</td>
<td>11.28</td>
</tr>
<tr>
<td>19</td>
<td>A</td>
<td>54.70</td>
<td>43.10</td>
<td>0.79</td>
<td>10.01</td>
<td>3003.27</td>
<td>1.04</td>
<td>3.52</td>
<td>84.60</td>
<td>4.41</td>
<td>102.22</td>
<td>10.57</td>
</tr>
<tr>
<td>20</td>
<td>A</td>
<td>54.70</td>
<td>43.00</td>
<td>0.79</td>
<td>10.68</td>
<td>2996.31</td>
<td>1.04</td>
<td>3.76</td>
<td>90.22</td>
<td>4.70</td>
<td>109.01</td>
<td>11.28</td>
</tr>
<tr>
<td>21</td>
<td>A</td>
<td>54.70</td>
<td>41.50</td>
<td>0.76</td>
<td>8.68</td>
<td>2891.78</td>
<td>1.03</td>
<td>3.03</td>
<td>72.74</td>
<td>3.79</td>
<td>87.89</td>
<td>9.09</td>
</tr>
<tr>
<td>22</td>
<td>A</td>
<td>54.70</td>
<td>46.10</td>
<td>0.84</td>
<td>8.01</td>
<td>3212.32</td>
<td>1.06</td>
<td>2.86</td>
<td>68.73</td>
<td>3.58</td>
<td>83.05</td>
<td>8.59</td>
</tr>
<tr>
<td>23</td>
<td>A</td>
<td>54.70</td>
<td>43.20</td>
<td>0.79</td>
<td>13.36</td>
<td>3010.24</td>
<td>1.04</td>
<td>4.71</td>
<td>112.97</td>
<td>5.88</td>
<td>136.51</td>
<td>14.12</td>
</tr>
<tr>
<td>24</td>
<td>A</td>
<td>54.70</td>
<td>44.00</td>
<td>0.80</td>
<td>8.68</td>
<td>3065.99</td>
<td>1.05</td>
<td>3.07</td>
<td>73.70</td>
<td>3.84</td>
<td>89.06</td>
<td>9.21</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>54.7</td>
<td>42.66</td>
<td>0.78</td>
<td>10.21</td>
<td>2972.50</td>
<td>1.04</td>
<td>3.59</td>
<td>86.13</td>
<td>4.49</td>
<td>104.07</td>
<td>10.77</td>
</tr>
</tbody>
</table>

* Bieniawski Criterion  
** Hassani Criterion
### Table 3.15 Results of Point load tests (specimens from Marulan Quarry) based on the equation $I_S(50) = \frac{F \cdot P}{D^2}$

<table>
<thead>
<tr>
<th>Number</th>
<th>Type</th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>(L/D)</th>
<th>P (KN)</th>
<th>$D^2$</th>
<th>F</th>
<th>$I_S(50)$</th>
<th>$\sigma_c$</th>
<th>$\sigma_t$</th>
<th>$\sigma_c^{**}$</th>
<th>$\sigma_t^{**}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>D</td>
<td>54.7</td>
<td>69.8</td>
<td>1.28</td>
<td>14.6</td>
<td>2992.09</td>
<td>1.05</td>
<td>5.10</td>
<td>122.49</td>
<td>6.38</td>
<td>148.01</td>
<td>15.31</td>
</tr>
<tr>
<td>2</td>
<td>D</td>
<td>54.7</td>
<td>68.2</td>
<td>1.25</td>
<td>10.68</td>
<td>2992.09</td>
<td>1.05</td>
<td>3.73</td>
<td>89.60</td>
<td>4.67</td>
<td>108.27</td>
<td>11.20</td>
</tr>
<tr>
<td>3</td>
<td>D</td>
<td>54.6</td>
<td>68.6</td>
<td>1.26</td>
<td>10.68</td>
<td>2981.16</td>
<td>1.04</td>
<td>3.74</td>
<td>89.85</td>
<td>4.68</td>
<td>108.57</td>
<td>11.23</td>
</tr>
<tr>
<td>4</td>
<td>D</td>
<td>54.6</td>
<td>68.7</td>
<td>1.26</td>
<td>5.35</td>
<td>2981.16</td>
<td>1.04</td>
<td>1.88</td>
<td>45.01</td>
<td>2.34</td>
<td>54.38</td>
<td>5.63</td>
</tr>
<tr>
<td>5</td>
<td>D</td>
<td>54.6</td>
<td>68.3</td>
<td>1.25</td>
<td>4.06</td>
<td>2981.16</td>
<td>1.04</td>
<td>1.42</td>
<td>34.16</td>
<td>1.78</td>
<td>41.27</td>
<td>4.27</td>
</tr>
<tr>
<td>6</td>
<td>D</td>
<td>54.6</td>
<td>72</td>
<td>1.32</td>
<td>10.68</td>
<td>2981.16</td>
<td>1.04</td>
<td>3.74</td>
<td>89.85</td>
<td>4.68</td>
<td>108.57</td>
<td>11.23</td>
</tr>
<tr>
<td>7</td>
<td>D</td>
<td>54.6</td>
<td>70.1</td>
<td>1.28</td>
<td>12.02</td>
<td>2981.16</td>
<td>1.04</td>
<td>4.21</td>
<td>101.12</td>
<td>5.27</td>
<td>122.19</td>
<td>12.64</td>
</tr>
<tr>
<td>8</td>
<td>D</td>
<td>54.7</td>
<td>70.6</td>
<td>1.29</td>
<td>13.36</td>
<td>2992.09</td>
<td>1.05</td>
<td>4.67</td>
<td>112.09</td>
<td>5.84</td>
<td>135.44</td>
<td>14.01</td>
</tr>
<tr>
<td>9</td>
<td>D</td>
<td>54.7</td>
<td>68.1</td>
<td>1.24</td>
<td>8.01</td>
<td>2992.09</td>
<td>1.05</td>
<td>2.80</td>
<td>67.20</td>
<td>3.50</td>
<td>81.20</td>
<td>8.40</td>
</tr>
<tr>
<td>10</td>
<td>D</td>
<td>54.7</td>
<td>70.4</td>
<td>1.29</td>
<td>6.68</td>
<td>2992.09</td>
<td>1.05</td>
<td>2.34</td>
<td>56.04</td>
<td>2.92</td>
<td>67.72</td>
<td>7.01</td>
</tr>
<tr>
<td>11</td>
<td>D</td>
<td>54.7</td>
<td>69.5</td>
<td>1.27</td>
<td>6.68</td>
<td>2992.09</td>
<td>1.05</td>
<td>2.34</td>
<td>56.04</td>
<td>2.92</td>
<td>67.72</td>
<td>7.01</td>
</tr>
<tr>
<td>12</td>
<td>D</td>
<td>54.6</td>
<td>70.3</td>
<td>1.29</td>
<td>9.35</td>
<td>2981.16</td>
<td>1.04</td>
<td>3.28</td>
<td>78.66</td>
<td>4.10</td>
<td>95.05</td>
<td>9.83</td>
</tr>
<tr>
<td>13</td>
<td>D</td>
<td>54.6</td>
<td>70</td>
<td>1.28</td>
<td>13.35</td>
<td>2981.16</td>
<td>1.04</td>
<td>4.68</td>
<td>112.31</td>
<td>5.85</td>
<td>135.71</td>
<td>14.04</td>
</tr>
<tr>
<td>14</td>
<td>D</td>
<td>54.6</td>
<td>68.5</td>
<td>1.25</td>
<td>10.5</td>
<td>2981.16</td>
<td>1.04</td>
<td>3.68</td>
<td>88.33</td>
<td>4.60</td>
<td>106.74</td>
<td>11.04</td>
</tr>
<tr>
<td>15</td>
<td>D</td>
<td>54.7</td>
<td>71</td>
<td>1.30</td>
<td>8.50</td>
<td>2992.09</td>
<td>1.05</td>
<td>2.97</td>
<td>71.31</td>
<td>3.71</td>
<td>86.17</td>
<td>8.91</td>
</tr>
<tr>
<td>16</td>
<td>D</td>
<td>54.7</td>
<td>68.6</td>
<td>1.25</td>
<td>4.06</td>
<td>2992.09</td>
<td>1.05</td>
<td>1.42</td>
<td>34.06</td>
<td>1.77</td>
<td>41.16</td>
<td>4.26</td>
</tr>
<tr>
<td>17</td>
<td>D</td>
<td>54.7</td>
<td>68.2</td>
<td>1.25</td>
<td>8.01</td>
<td>2992.09</td>
<td>1.05</td>
<td>2.80</td>
<td>67.20</td>
<td>3.50</td>
<td>81.20</td>
<td>8.40</td>
</tr>
<tr>
<td>Mean value</td>
<td>-</td>
<td>54.65</td>
<td>69.46</td>
<td>1.27</td>
<td>9.21</td>
<td>2986.95</td>
<td>1.05</td>
<td>3.22</td>
<td>77.37</td>
<td>4.03</td>
<td>93.49</td>
<td>9.67</td>
</tr>
</tbody>
</table>

* Bieniawaski criterion  
** Hassania criterion
Table 3.16  Results of point load tests (specimens from Dunmore Quarry) based on the equation $I_{50} = F.P/D_e^2$.

<table>
<thead>
<tr>
<th>Number</th>
<th>Type</th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>(L/D)</th>
<th>P (KN)</th>
<th>$D_e^2$</th>
<th>F</th>
<th>$I_{50}$ (50)</th>
<th>$\sigma_c^*$</th>
<th>$\sigma_t^*$</th>
<th>$\sigma_c^{**}$</th>
<th>$\sigma_t^{**}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>54.70</td>
<td>48.00</td>
<td>0.88</td>
<td>27.38</td>
<td>3344.71</td>
<td>1.07</td>
<td>8.74</td>
<td>209.76</td>
<td>10.93</td>
<td>253.46</td>
<td>26.22</td>
</tr>
<tr>
<td>2</td>
<td>A</td>
<td>54.70</td>
<td>47.90</td>
<td>0.88</td>
<td>27.38</td>
<td>3337.75</td>
<td>1.07</td>
<td>8.75</td>
<td>210.10</td>
<td>10.94</td>
<td>253.87</td>
<td>26.26</td>
</tr>
<tr>
<td>3</td>
<td>A</td>
<td>54.70</td>
<td>47.20</td>
<td>0.86</td>
<td>25.37</td>
<td>3288.97</td>
<td>1.06</td>
<td>8.20</td>
<td>196.91</td>
<td>10.26</td>
<td>237.94</td>
<td>24.61</td>
</tr>
<tr>
<td>4</td>
<td>A</td>
<td>54.70</td>
<td>47.00</td>
<td>0.86</td>
<td>28.71</td>
<td>3275.03</td>
<td>1.06</td>
<td>9.32</td>
<td>223.57</td>
<td>11.64</td>
<td>270.15</td>
<td>27.95</td>
</tr>
<tr>
<td>5</td>
<td>A</td>
<td>54.70</td>
<td>48.10</td>
<td>0.88</td>
<td>25.37</td>
<td>3351.68</td>
<td>1.07</td>
<td>8.09</td>
<td>194.05</td>
<td>10.11</td>
<td>234.48</td>
<td>24.26</td>
</tr>
<tr>
<td>6</td>
<td>A</td>
<td>54.70</td>
<td>47.00</td>
<td>0.86</td>
<td>29.38</td>
<td>3275.03</td>
<td>1.06</td>
<td>9.53</td>
<td>228.79</td>
<td>11.92</td>
<td>276.45</td>
<td>28.60</td>
</tr>
<tr>
<td>7</td>
<td>A</td>
<td>54.70</td>
<td>47.60</td>
<td>0.87</td>
<td>27.37</td>
<td>3316.84</td>
<td>1.07</td>
<td>8.79</td>
<td>211.05</td>
<td>10.99</td>
<td>255.02</td>
<td>26.38</td>
</tr>
<tr>
<td>8</td>
<td>A</td>
<td>54.70</td>
<td>47.10</td>
<td>0.86</td>
<td>24.70</td>
<td>3282.00</td>
<td>1.06</td>
<td>8.00</td>
<td>192.03</td>
<td>10.00</td>
<td>232.03</td>
<td>24.00</td>
</tr>
<tr>
<td>9</td>
<td>A</td>
<td>54.70</td>
<td>48.00</td>
<td>0.88</td>
<td>30.71</td>
<td>3275.03</td>
<td>1.06</td>
<td>9.96</td>
<td>239.15</td>
<td>12.46</td>
<td>288.97</td>
<td>29.89</td>
</tr>
<tr>
<td>10</td>
<td>A</td>
<td>54.70</td>
<td>47.00</td>
<td>0.86</td>
<td>26.04</td>
<td>3337.75</td>
<td>1.07</td>
<td>8.33</td>
<td>199.82</td>
<td>10.41</td>
<td>241.45</td>
<td>24.98</td>
</tr>
<tr>
<td>11</td>
<td>A</td>
<td>54.70</td>
<td>48.00</td>
<td>0.88</td>
<td>26.04</td>
<td>3344.71</td>
<td>1.07</td>
<td>8.31</td>
<td>199.50</td>
<td>10.39</td>
<td>241.06</td>
<td>24.94</td>
</tr>
<tr>
<td>12</td>
<td>A</td>
<td>54.70</td>
<td>47.00</td>
<td>0.86</td>
<td>26.71</td>
<td>3344.71</td>
<td>1.07</td>
<td>8.53</td>
<td>204.63</td>
<td>10.66</td>
<td>247.26</td>
<td>25.58</td>
</tr>
<tr>
<td>13</td>
<td>A</td>
<td>54.70</td>
<td>47.20</td>
<td>0.86</td>
<td>26.71</td>
<td>3288.97</td>
<td>1.06</td>
<td>8.64</td>
<td>207.31</td>
<td>10.80</td>
<td>250.50</td>
<td>25.91</td>
</tr>
<tr>
<td>14</td>
<td>A</td>
<td>54.70</td>
<td>48.20</td>
<td>0.88</td>
<td>26.04</td>
<td>3358.65</td>
<td>1.07</td>
<td>8.29</td>
<td>198.86</td>
<td>10.36</td>
<td>240.28</td>
<td>24.86</td>
</tr>
<tr>
<td>15</td>
<td>A</td>
<td>54.70</td>
<td>47.00</td>
<td>0.86</td>
<td>26.71</td>
<td>3275.03</td>
<td>1.06</td>
<td>8.67</td>
<td>208.00</td>
<td>10.83</td>
<td>251.33</td>
<td>26.00</td>
</tr>
<tr>
<td>16</td>
<td>A</td>
<td>54.70</td>
<td>47.90</td>
<td>0.88</td>
<td>28.04</td>
<td>3337.75</td>
<td>1.07</td>
<td>8.97</td>
<td>215.17</td>
<td>11.21</td>
<td>259.99</td>
<td>26.90</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>54.70</td>
<td>47.57</td>
<td>0.87</td>
<td>27.04</td>
<td>3314.66</td>
<td>1.07</td>
<td>8.69</td>
<td>208.67</td>
<td>10.87</td>
<td>252.14</td>
<td>26.08</td>
</tr>
</tbody>
</table>

* Bieniawski criterion

** Hassani Criterion
Table 3.17 Results of Point load test (specimens from Dunmore Quarry) based on the equation $I_S(50) = F.P/De^2$

<table>
<thead>
<tr>
<th>Number</th>
<th>Type</th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>(L/D)</th>
<th>P (KN)</th>
<th>De$^2$</th>
<th>F</th>
<th>$I_S (50)$</th>
<th>$\sigma_c*$</th>
<th>$\sigma_t*$</th>
<th>$\sigma_c^{**}$</th>
<th>$\sigma_t^{**}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>D</td>
<td>54.7</td>
<td>71.3</td>
<td>1.30</td>
<td>22.7</td>
<td>2992.09</td>
<td>1.05</td>
<td>7.94</td>
<td>190.45</td>
<td>9.92</td>
<td>230.12</td>
<td>23.81</td>
</tr>
<tr>
<td>2</td>
<td>D</td>
<td>54.7</td>
<td>70.4</td>
<td>1.29</td>
<td>21.36</td>
<td>2992.09</td>
<td>1.05</td>
<td>7.47</td>
<td>179.20</td>
<td>9.33</td>
<td>216.54</td>
<td>22.40</td>
</tr>
<tr>
<td>3</td>
<td>D</td>
<td>54.7</td>
<td>71.1</td>
<td>1.30</td>
<td>26.71</td>
<td>2992.09</td>
<td>1.05</td>
<td>9.34</td>
<td>224.09</td>
<td>11.67</td>
<td>270.77</td>
<td>28.01</td>
</tr>
<tr>
<td>4</td>
<td>D</td>
<td>54.6</td>
<td>70.6</td>
<td>1.29</td>
<td>24.03</td>
<td>2981.16</td>
<td>1.04</td>
<td>8.42</td>
<td>202.16</td>
<td>10.53</td>
<td>244.27</td>
<td>25.27</td>
</tr>
<tr>
<td>5</td>
<td>D</td>
<td>54.6</td>
<td>71.2</td>
<td>1.30</td>
<td>25.37</td>
<td>2981.16</td>
<td>1.04</td>
<td>8.89</td>
<td>213.43</td>
<td>11.12</td>
<td>257.90</td>
<td>26.68</td>
</tr>
<tr>
<td>6</td>
<td>D</td>
<td>54.6</td>
<td>70.3</td>
<td>1.29</td>
<td>26.71</td>
<td>2981.16</td>
<td>1.04</td>
<td>9.36</td>
<td>224.70</td>
<td>11.70</td>
<td>271.52</td>
<td>28.09</td>
</tr>
<tr>
<td>7</td>
<td>D</td>
<td>54.6</td>
<td>70.2</td>
<td>1.29</td>
<td>26.71</td>
<td>2981.16</td>
<td>1.04</td>
<td>9.36</td>
<td>224.70</td>
<td>11.70</td>
<td>271.52</td>
<td>28.09</td>
</tr>
<tr>
<td>8</td>
<td>D</td>
<td>54.7</td>
<td>71.2</td>
<td>1.30</td>
<td>24.03</td>
<td>2992.09</td>
<td>1.05</td>
<td>8.40</td>
<td>201.60</td>
<td>10.50</td>
<td>243.60</td>
<td>25.20</td>
</tr>
<tr>
<td>9</td>
<td>D</td>
<td>54.7</td>
<td>69.7</td>
<td>1.27</td>
<td>25.37</td>
<td>2992.09</td>
<td>1.05</td>
<td>8.87</td>
<td>212.85</td>
<td>11.09</td>
<td>257.19</td>
<td>26.61</td>
</tr>
<tr>
<td>10</td>
<td>D</td>
<td>54.6</td>
<td>70.6</td>
<td>1.29</td>
<td>13.35</td>
<td>2981.16</td>
<td>1.04</td>
<td>4.68</td>
<td>112.31</td>
<td>5.85</td>
<td>135.71</td>
<td>14.04</td>
</tr>
<tr>
<td>11</td>
<td>D</td>
<td>54.7</td>
<td>70.1</td>
<td>1.28</td>
<td>26.71</td>
<td>2992.09</td>
<td>1.05</td>
<td>9.34</td>
<td>224.09</td>
<td>11.67</td>
<td>270.77</td>
<td>28.01</td>
</tr>
<tr>
<td>12</td>
<td>D</td>
<td>54.6</td>
<td>71.2</td>
<td>1.30</td>
<td>28.71</td>
<td>2981.16</td>
<td>1.04</td>
<td>10.06</td>
<td>241.53</td>
<td>12.58</td>
<td>291.85</td>
<td>30.19</td>
</tr>
<tr>
<td>13</td>
<td>D</td>
<td>54.6</td>
<td>70.7</td>
<td>1.29</td>
<td>26.71</td>
<td>2981.16</td>
<td>1.04</td>
<td>9.36</td>
<td>224.70</td>
<td>11.70</td>
<td>271.52</td>
<td>28.09</td>
</tr>
<tr>
<td>14</td>
<td>D</td>
<td>54.6</td>
<td>69.4</td>
<td>1.27</td>
<td>26.71</td>
<td>2981.16</td>
<td>1.04</td>
<td>9.36</td>
<td>224.70</td>
<td>11.70</td>
<td>271.52</td>
<td>28.09</td>
</tr>
<tr>
<td>15</td>
<td>D</td>
<td>54.7</td>
<td>70.3</td>
<td>1.29</td>
<td>28.04</td>
<td>2992.09</td>
<td>1.05</td>
<td>9.80</td>
<td>235.25</td>
<td>12.25</td>
<td>284.26</td>
<td>29.41</td>
</tr>
<tr>
<td>16</td>
<td>D</td>
<td>54.7</td>
<td>69.9</td>
<td>1.28</td>
<td>28.04</td>
<td>2992.09</td>
<td>1.05</td>
<td>9.80</td>
<td>235.25</td>
<td>12.25</td>
<td>284.26</td>
<td>29.41</td>
</tr>
<tr>
<td>17</td>
<td>D</td>
<td>54.7</td>
<td>68.2</td>
<td>1.25</td>
<td>26.71</td>
<td>2992.09</td>
<td>1.05</td>
<td>9.34</td>
<td>224.09</td>
<td>11.67</td>
<td>270.77</td>
<td>28.01</td>
</tr>
<tr>
<td>18</td>
<td>D</td>
<td>54.7</td>
<td>69.1</td>
<td>1.26</td>
<td>24.70</td>
<td>2992.09</td>
<td>1.05</td>
<td>8.63</td>
<td>207.23</td>
<td>10.79</td>
<td>250.40</td>
<td>25.90</td>
</tr>
<tr>
<td>19</td>
<td>D</td>
<td>54.7</td>
<td>68.5</td>
<td>1.25</td>
<td>28.71</td>
<td>2992.09</td>
<td>1.05</td>
<td>10.04</td>
<td>240.87</td>
<td>12.55</td>
<td>291.05</td>
<td>30.11</td>
</tr>
<tr>
<td>Mean</td>
<td>-</td>
<td>54.66</td>
<td>70.21</td>
<td>1.28</td>
<td>25.34</td>
<td>2987.49</td>
<td>1.05</td>
<td>8.87</td>
<td>212.80</td>
<td>11.08</td>
<td>257.13</td>
<td>26.60</td>
</tr>
</tbody>
</table>

* Bieniawaski Criterion
** Hassani Criterion
3.12. LABORATORY TESTS CORRELATIONS FOR HARD ROCKS

An statistical analysis of the results of the laboratory test results were carried out in order to find relationship between different type of test results from different types of hard rocks. According to the results of tests and their relationship some of the regression methods were examined. Linear, power and exponential regression were used and the relationship between the test results were determined. The results of correlation are described as follows:

3.12.1 Uniaxial Compressive Strength and Diametral Point Load Index Test

As it was expressed in the previous sections, the uniaxial compressive strength test is one of the most commonly used method for considering of the strength characteristics of intact rock. Most of the engineering properties of rock can be determined from the results of the uniaxial compressive strength tests. The results of uniaxial compressive strength (UCS) were presented in the previous sections.

The correlation of diametral point load index and the results of uniaxial compressive strength for four types of hard rock from different open pit mines is given in Figure 3.21. Test results in a power regression shows very high correlation coefficient of $r^2 = 0.999$. The derived correlation equations calculate uniaxial compressive strength (UCS) from the following equation:

$$
\sigma_c = 34.69 \ I_{sd}^{0.710}
$$

(3.16)

Where

$\sigma_c =$ Uniaxial Compressive Strength

$I_{sd} =$ Point load index
Figure 3.21 power regression between uniaxial compressive strength and diametral point load index test for hard rocks

The linear relationship of the uniaxial compressive strength (UCS) and diametral point load index also has a high corresponding correlation coefficient of $r^2 = 0.919$. This relationship is given in the following equation:

$$\sigma_c = 34.695 I_{sd}^{0.710} \quad r^2 = 0.999$$

Where;

$\sigma_c =$ Uniaxial Compressive Strength

$I_{sd} =$ Diametral Point load Index

The plot of the uniaxial compressive strength and the diametral point load test is given in Figure 3.22.
CHAPTER THREE  Data Acquisition and Estimation of Rock Mass Strength

250

Diametral Point Load Test, MPa

Figure 3.22 Linear correlation between uniaxial compressive strength and diametral point load index test for different types of hard rocks.

3.12.2 Uniaxial Compressive Strength and Axial Point Load Test:

The linear relationship of the uniaxial compressive strength (UCS) and the axial point load index is given in Figure 3.23. Test results have a good corresponding correlation coefficient of $r^2 = 0.946$. The uniaxial compressive strength (UCS) can be calculated from the following equation;

$$
\sigma_c = 9.399I_{sa} + 63.630
$$

(3.18)

Where;

$\sigma_c = $ Uniaxial Compressive Strength

$I_{sa} = $ Axial Point Load Index
Figure 3.23 Correlation between uniaxial compressive strength and axial point load index for different types of hard rocks

The power regression of the uniaxial compressive strength (UCS) and axial point load index test (Figure 3.24) shows very high corresponding correlation coefficient of $r^2 = 0.999$. As a result, uniaxial compressive strength (UCS) for hard rock specimens can be calculated from the following equation:

$$\sigma_c = 33.370 \, I_{sa}^{0.673}$$  \hspace{1cm} (3.19)

Where;

$\sigma_c =$ Uniaxial Compressive Strength

$I_{sa} =$ Axial point load Index
3.12.3 Uniaxial Compressive Strength and Tensile Strength Test

The linear relationship of the uniaxial compressive strength (UCS) with the Brazilian test results shows a very good and corresponding correlation coefficient of \( r^2 = 0.963 \). This relationship is given in Figure 3.25. The following equation expressed the relationship between uniaxial compressive strength and the tensile strength of different types of hard rocks;

\[
\sigma_c = 15.300 \sigma_t + 12.028 \tag{3.20}
\]

Where:

\( \sigma_c \) = Uniaxial Compressive Strength

\( \sigma_t \) = Tensile Strength
Figure 3.25 Linear regression between uniaxial compressive strength and tensile strength for different types of hard rocks.

In addition an exponential relationship between uniaxial compressive strength ($\sigma_c$) and Brazilian test results ($\sigma_t$) shows a very high corresponding coefficient of $r^2 = 0.974$. This relationship is given in Figure 3.26 and it is expressed with the following equation;

$$\sigma_c = 48.255 \times 10^{-0.052} \sigma_t$$  \hspace{1cm} (3.21)

where;

$\sigma_c = \text{Uniaxial Compressive Strength}$

$\sigma_t = \text{Tensile Strength}$
3.12.4 Tensile Strength Test and Diametral Point Load Index

The power regression relationship of the Brazilian test with the diametral point load index indicated that there is a high correlation coefficient of $r^2 = 0.927$. The equation derived from this correlation is given as follows:

$$\sigma_t = 1.460 \cdot I_{sd}^{0.845}$$  \hspace{1cm} (3.22)

Where;

$\sigma_t$ = Tensile Strength

$I_{sd}$ = Diametral Point Load Index

This relationship is given in Figure 3.27. In addition a linear regression between tensile strength and diametral point load index for hard rocks shows good corresponding correlation coefficient of $r^2 = 0.925$. This relationship is given in Figure 3.28. The Equation 3.25 expressed the relationship between axial point load index and tensile strength for hard rocks.

Figure 3.26 Exponential regression between uniaxial compressive strength and tensile strength for different types of hard rocks.
\[
\sigma_t = 0.846 I_{sd} + 1.972
\]

(3.23)

Where;

\( \sigma_t \) = Tensile Strength
\( I_{sd} \) = Diametral Point Load Index

Figure 3.27 Power regression between tensile strength and diametral point load index test for different types of hard rocks.

### 3.12.5 Tensile strength and Axial Point Load Index Test

The Linear relationship of the Brazilian test and the axial point load test has a high corresponding correlation coefficient of \( r^2 = 0.921 \). This relationship is expressed by the following equation.

\[
\sigma_t = 0.570 I_{sa} + 3.859
\]

(3.24)

where

\( \sigma_t \) = Tensile Strength
\( I_{sa} \) = Axial Point Load Index

This relationship is given in Figure 3.29.
Diametral Point Load Test, MPa

Figure 3.28 Linear regression between tensile strength and diametral point load index for different types of hard rocks from different open pit mines.

\[
\sigma_t = 0.864 \text{Isd} + 1.972 \quad r^2 = 0.925
\]

Axial Point Load Test, MPa

Figure 3.29 Linear regression between tensile strength and axial point load index test for different types of hard rocks.

\[
\sigma_t = 0.570 \text{Isa} + 3.859 \quad r^2 = 0.921
\]
3.12.6 Diametral and Axial Point Load Test

The correlation of measured diametral point load test with the axial point load index is
given in Figure 3.30. As a result the diametral point load index can be calculated from
the linear relationship and from the following equation

\[ I_{sd} = 0.661 I_{sa} + 2.347 \]  \hspace{1cm} (3.25)

This relationship also has a high correlation coefficient of \( r^2 = 0.983 \) and it can be used
for calculation of diametral point load index from the axial point load index test.

In addition a power regression from the diametral and axial point load index show a
very high corresponding correlation coefficient of 0.997. This relationship is given in
the following equation

\[ I_{sd} = 1.724 I_{sa}^{0.728} \]  \hspace{1cm} (3.26)

Figure 3.30 Linear regression between diametral and axial point load index test for
different types of hard rocks
3.13 SCALE EFFECT ON THE STRENGTH AND ELASTIC MODULUS OF ROCKS

In the last three decades, many attempts have been made to establish a quantitative relationship between laboratory test results and the in-situ properties of rock mass. Many attempts have also been made to make a correlation between Rock Quality Designation (RQD) and the ratio ($E_M/E_l$) of rock mass modulus $E_M$ to laboratory modulus $E_l$. In this regard Coon and Merritt (1970) proposed a correlation factor of 0.544 for this relationship. In addition, Heuze (1980) stated that the moduli values measured in the laboratory are, on the average, 2.5 times higher than the in-situ values.

In general, the ratio between the in-situ modulus and laboratory test results is between 0.2 to 0.6. Bieniawski (1978) proposed a correlation between the in-situ modulus and Rock Mass Rating (RMR) for a good quality rock mass i.e. RMR>50.

$$E_M = 2\cdot RMR - 100$$  \hspace{1cm} (3.27)

where;

$E_M = \text{In situ rock mass modulus (GPa)}$
Serafim and Pereira (1983) have shown that there is a non linear relationship between RMR and \( E_M \) for RMR < 50. They provided results in the range RMR > 50 and proposed a new correlation as follow:

\[ E_M = 10^{(RMR-10)^{0.40}} \]  

(3.28)

A correlation between laboratory compressive strength and in-situ strength has been proposed by Hoek and Brown (1980). The equation 3.29 expressed this relationship. The correlation for rock mass strength is as follows:

\[ \frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \sqrt{m \frac{\sigma_3}{\sigma_c} + s} \]  

(3.29)

where

- \( \sigma_1 \) is major principal stress at failure
- \( \sigma_3 \) is applied minor principal stress
- \( \sigma_c \) is uniaxial compressive strength of the material and
- \( m \) and \( s \) are constants dependent upon the properties of the rock and the extent to which it has been fractured by being subjected to \( \sigma_1 \) and \( \sigma_3 \).

\[ \sigma_{1n} = \sigma_{3n} + \sqrt{m. \sigma_{3n} + s} \]  

(3.30)

In this equation \( m \) and \( s \) are depends on the rock properties and the extent to which the rock mass has been fractured by being subjected to principal stress and confining pressure i.e. \( \sigma_1 \) and \( \sigma_3 \) respectively. Priest and Brown proposed the following equations for estimation of \( m \) and \( s \):

\[ s = \exp\left(\frac{RMR - 100}{6}\right) \]  

(3.31)

\[ m = m_i \exp\left(\frac{RMR - 100}{14}\right) \]  

(3.32)

where

- \( m_i \) is the rock constant \( m \) for intact rock determined from laboratory triaxial tests.

Trueman et al (1992) proposed that the uniaxial compressive strength (UCS) of the rock mass (\( \sigma_{cm} \)) can be calculated from the following relationship derived from the Hoek and Brown failure criterion:
where $\sigma_c$ is the uniaxial compressive strength of laboratory sized intact samples.

It should be noted that for intact rock $s = 1$ and $m = m_i$ which is determined from a fit of the Equation 3.29 to triaxial test results derived from laboratory specimens.

For rock masses, the constants $m$ and $s$ are related to the basic (unadjusted) rock mass rating (RMR), as follows (Hoek and Brown, 1988). For undisturbed rock masses (smooth-blasted or machine-bored excavations):

$$m = m_i \exp[(RMR - 100)/28]$$

(3.34)

$$s = \exp[(RMR - 100)/9]$$

(3.35)

For disturbed rock masses (slopes or blast-damaged excavations):

$$m = m_i \exp[(RMR - 100)/14]$$

(3.36)

$$s = \exp[(RMR - 100)/6]$$

(3.37)

The rock failure criterion proposed by Bieniawski (1974) was modified by Yudhbir (1983) in respect to the constant $A$, as follows:

$$\frac{\sigma_1}{\sigma_c} = A + B \left[ \frac{\sigma_3}{\sigma_c} \right]^a$$

(3.38)

where

$a = 0.75$ and $B$ depends on rock type and it is proposed $B = 1$ for limestone, $B = 2$; siltstone and mudstone, $B = 3$; sandstone and quartzite, $B = 4$; norite and granite; $B = 5$. In the Bieniawski criterion, $A = 1$ for intact rock and rock masses while Yudhbir suggested that $A$ is a function of rock mass quality as follows:

$$A = 20. m \ (0.0765 \ RMR - 7.65)$$

(3.39)

### 3.14 CONCLUSIONS

This chapter deals with the methods of sampling, sample preparation and laboratory testing of intact rock. Moreover, the engineering properties of different types of rock in relation to the designing of rock slope faces within a rock mass were also considered. Laboratory tests were carried out to investigate the effect of physical and mechanical properties of intact.
Different types of laboratory and index tests were carried out on the samples of porphyry, limestone, and basalt. The results of the tests are given in different way which shows the behaviour of the intact rocks under loading in order to evaluate the engineering properties of intact rock for the aim of rock slope stability in open pit mines. The following results have been concluded from the laboratory and index tests on different type of rocks.

- Inherent properties of rocks are very important strength parameters which should be included in a laboratory testing program in any engineering rock project.

- Most of the information used for designing in rock material can be achieved by laboratory tests and also the index tests. In the other words, one of the advantages of the laboratory tests is applying these results for the rock mass classification and slope design in open pit mines.

- Point load and Brazilian Index tests are very simple and valuable tests for the determining of rock tensile strength and other mechanical properties of rocks. In a short time and with a low budget they can give valuable information about the rock properties.

- Point load index test is useful index test for hard rock samples. Due to the experience it seems that in a large number of point load tests carried out on one type of hard rock, the results are very close together.

- The values of uniaxial compressive strength obtained by point load index tests using Hassani et al (1980) criterion are higher than the results which are directly obtained by uniaxial compressive strength tests.

- The uniaxial compressive strength tests were carried out on a large number of hard rock samples to evaluate the engineering properties of intact rock and it was found that the stress-strain curves for all samples from different types of hard rock shows a linear elastic behaviour.

- Because of brittleness of the samples it was not possible to obtain the post failure behaviour of rock samples under loading. It may be noted that most of the samples were failed immediately after reaching the peak strength.

- The values of tensile strength obtained by Brazilian tests for different types of rock varies between 1/10 to 1/20 of the values of uniaxial compressive strength.

- The values of tensile strength obtained by diametral point load tests for different types of rock varies between 1/15 to 1/20 of the values of uniaxial compressive strength.
Chapter 4
The Shear Strength of Rock and Rock Joints
CHAPTER FOUR

THE SHEAR STRENGTH OF ROCK AND ROCK JOINTS

4.1 INTRODUCTION

The presence of joints can significantly affect the mechanical behaviour of rock masses by reducing their capability to support shear and tensile loading. When a natural in-situ joint is disturbed from its initial condition by excavation and/or loading, it typically undergoes a response that is rich in phenomena including frictional sliding, dilatancy, damage of asperity surfaces and particularly for dynamic loading or a sequence of construction or excavation events, possible cycle behaviour. The full history of the joint response contributes to the overall deformation of the rock mass and therefore must be accurately understood and quantified for meaningful numerical simulations, (Huang et al, 1993).

This chapter deals with the description of most important engineering features relating to shear strength of rocks and rock joints. In addition, it contains a description of the shear strength parameters of discontinuities and their influence on the designing of open pit mines in rock masses. In mining operations, rock engineering problems other than those involving only fracture of intact rock, where the shear strength of the discontinuities has a great effect on the behaviour of the rock mass.
The direct shear test is one of the most commonly used methods for testing the shear strength of discontinuities in rock engineering. The direct shear test is a procedure in which a specimen is confined within rigid, fixed rotation frames and caused to shear on a plane. This test measures peak and residual direct shear strength as a function of stress normal to the shear plane. For the shear strength of fractures and fault gouge, the direct shear test is recommended as it is a simulator of field conditions (Call, 1992).

The direction of the strength of rock joints is similar in many respects to intact rock strength (Goodman, 1980; Brady and Brown, 1985; Einstein and Dowding, 1989), both are based on experimental observations and the practical need for simple mathematical forms, quoted by Parisau, 1992).

Studies of the shear strength of jointed rock masses have been carried out by many authors, e.g. Patton (1966); Ladanyi & Archambault (1970); Moretto et al. (1970); Barton (1973; 1976, and 1982); Brawner et al (1971, 1986); John (1974); Hoek (1977); Chowdhury (1978); Franklin (1970, 1989); Singh et al (1985, 86, 87, and 89); Hassani et al (1980); Hoek and Brown (1980); Bandis et al (1981); Bilgin et al (1990); Hoek (1990); Kulatilake (1992); Saeb and Amadei (1992); Brady and Brown; (1985, 1993); Au (1993).

The aim of the current testing program on rock joints is simulating the actual and field failure surface in the laboratory. Shear tests are mainly conducted on existing planes of discontinuities such as bedding planes, joints, foliations etc, and have the purpose of obtaining information about the stability of the system when subjected to loads imposed by the structure (Lama and Vutukuri, 1978).

It is common for shear test to be performed to obtain both the peak and residual angle of friction. For residual shear strength determination, a sample should only be tested in one direction, since test reversal does not occur in the field, experience is that the reversal procedure generally provides a lower residual strength than will actually develop for the field condition (Brawner et al, 1986).

During direct shear test the following four main parameters were measured.

- Normal stress
• Shear stress
• Normal displacement
• Shear displacement

Measuring these data made it possible to draw a shear stress-normal stress envelope and shear stress-shear displacement diagram that was used for the determination of the discontinuity friction angle and cohesive strength. The most important factors influencing friction between the joint surfaces are summarised by Lama and Vutukuri, (1978) as roughness of the surface, displacement history, normal load, water, and filling material. Their influence in detail is discussed in this chapter. Before continuing with the above discussion, it is necessary to explain the joint sampling and testing procedure, and the apparatus employed for testing the naturally jointed samples.

To obtain shear strength characteristics for use in slope design, some form of testing is usually required. This may be a very sophisticated laboratory or in-situ test depending on the nature of the problem being investigated, (Ozgenoglu, 1990). In addition, the shear and normal stiffness of the discontinuities can exert a controlling influence on the distribution of stresses and displacements within a discontinuous rock mass. These properties can be measured in the same tests as those used to determine discontinuity shear strengths (Brady and Brown, 1993).

Rock joints exhibit a wide spectrum of shear strengths under the low effective normal stress levels operating in most rock engineering problems. This is due to the strong influence of surface roughness and variable rock strength (Barton, 1976). The angle of friction is influenced by the surface roughness of the discontinuity of the rock and will vary with the direction. As a result, tests should only be performed on samples in the direction at which failure would occur in the field (Brawner et al, 1986).

A hard rock mass may be closely jointed or fractured and its shear strength in this condition is often significantly different from that of a single discontinuity (Chowdhury, 1978). The behaviour of a large rock mass depends on both the properties of the component intact material and the properties of the defects within the rock mass (Rosso, 1976). During a shear strength investigation the choice of an appropriate joint test-size is generally based on both economic and technical considerations. Generally,
the large scale conventional shear test involve high cost. Therefore, laboratory testing of small joint samples is a cheaper alternative method and is used in this research.

4.2 JOINT SAMPLING AND PREPARATION

The methods of obtaining joint samples include drilling through joints in the field, or producing artificial fractures in the laboratory by breaking specimens of the actual rock or reproducing the wall rock features in plaster (Goodman, 1974). In order to study the mechanical behaviour of joint sets several block samples containing natural joint sets were chosen from the three different sites (Mugga II, Marulan, and Dunmore Quarries). Before choosing the joint samples, several core specimens from intact rock were prepared for direct shear testing, but because of the diameter of the samples (NX) and high strength of the specimens it was not possible to do this test. However, in order to consider the mechanical behaviour of the joints, some new samples were prepared form the natural jointed rocks. Samples were cast in pattern-stone plaster, a very strong form of plaster. Samples containing natural joint sets were prepared as follows:

- The jointed rock samples were chosen with the orientation of coring normal to the discontinuities.
- The two halves of the samples were carefully bound together by a wire loop.
- The samples were prepared for testing in a shear box.

The shear box consisted of two components, the upper parts is fitted with a small hydraulic ram for applying normal loading and the lower part was fitted with two rams for applying horizontal shear forces. Different stages of preparation of samples and casting of natural joints used in direct shear test are presented in Figure 4.1.

The relationship between shear and normal force is purely frictional and is independent of any forms of cohesion, Patton, (1966). It should be noted that for unbounded concrete rock surfaces, such a relationship should seem quite reasonable but, based on a large number of laboratory tested samples carried out by Lam and Johnston, (1989), this independence from cohesion was established to be confirm for the planar interfaces with \( i = 0 \). In the case of \( i > 0 \) it was identified that some cohesive component was present.
CHAPTER FOUR
The Shear Strength of Rock and Rock Joints

Figure 4.1 Preparation of specimen for direct shear test

Wire loop
Natural discontinuity surface
Core samples with natural jointing
Plaster
Direct shear mould
Casting one half the specimen
Casting the second half of the specimen
Cut wire loop in testing time
Cast specimen ready to be placed in the shear box
The direct shear tests carried out by Ladanyi and Archambault, (1970), and Barton and Bandis, (1980) on real jointed samples indicated that, the behaviour of real joints is likely to be strongly influenced by the irregular nature of roughness (Kodikara and Johnston, 1994).

4.3 DIRECT SHEAR TESTING METHODS

There are two different types of direct shear testing machine available for examining the direct shear strength of rock and joint samples.

• Large direct shear testing machine

• Small direct shear testing machine

In this research a small direct shear box testing machine was used. Normal force was applied by using a hand hydraulic pump with a hand pump and second hydraulic pump was employed for applying the shear force. The direction of the shear force is parallel to the contact plane between to pieces of samples.

After preparation of the apparatus and making the hydraulic connection, the sample which had been cast in plaster was placed in to the lower section of the direct shear box, then the upper section was placed on the top of the specimen. After fixing the specimen in the direct shear box, the normal load was employed to the full value specified for the test and again the dial gauge was then adjusted to zero. Then the shear force was applied continuously to control the rate of shear displacement. Generally, 7 to 20 readings were taken before reaching to the peak strength.

4.4 SHEAR BEHAVIOUR OF JOINTED ROCK MASS

The shear behaviour of joints in jointed rock masses has a great effect on the overall shear strength of rock. In mining, where rock mechanics problems other than those involving the fracture of intact rock are encountered, the shear behaviour of the discontinuities will be important (Brady and Brown, 1993). Also the shear strength of the discontinuities has an influence on the conditions of slip on the major fractures, such
as faults and bedding or in the sliding of individual blocks from the boundaries of excavations. In addition, the distribution of stresses and displacements within a discontinuous rock mass is governed by the shear and normal stiffness of discontinuities.

A detailed geological description of the joint and the wall rock may allow a sufficiently accurate assumption of the magnitudes of the relevant physical properties or else the properties can be derived from a program of laboratory tests on natural or artificially simulated surfaces (Goodman, 1974).

One of the most surprising conclusions arrived at as a result of high pressure triaxial tests on intact rock is the apparent lack of correlation between the fracture strength of the intact rock and the frictional strength of the resulting fault (Barton, 1976). However, experience and studies under low effective normal stress levels indicates that the shear strength of joints can vary within relatively wide limits as indicated.

As the strength behaviour of the rock mass is concerned, much valuable effort has been made in recent years toward reaching a proper understanding of the phenomenon of rock mass failure under stress (Ladanyi and Archambault, 1970). The study resulted in establishing a first useful working hypothesis of the shear behaviour of a regularly jointed rock mass was introduced by Patton (1966). The shear strength of a rock joint sample can be considered to consist of two main components:

- A basic frictional component ($\phi_b$) which presents the minimum resistance between two flat surface of a rock joint.

- An additional resistance due to the presence of irregularities and roughness on the joint walls.

In this regard the effect of roughness of the surface may be recognised in the shear strength versus normal stress curve as an increase in the observed friction angle over the basic friction angle of the rock. The frictional resistance ($\phi_b$) developed between two flat rock surfaces is affected by the mineralogical composition of the material and the moisture content.
Newland and Allely (1957) explained that the shear resistance of non-planer rock joints seems to be related to the observed dilatant behaviour of granular materials such as sand. They developed following equation for the evaluation of the maximum shear strength.

\[ \tau = \sigma_n \tan (\phi_b + i) \]  

(4.1)

Patton, et al, (1966) and Goldstein, et al, (1966) used the above equation to present the shear strength of irregular rock surfaces and broken rock, when tested at low normal stresses. At high normal stresses it was assumed that the Coulomb relationship applied:

\[ \tau = C + \sigma_n \tan \phi \]  

(4.2)

The recognition that the shear strength of an irregular rock surface can be zero at zero normal stress represents a very big improvement over the earlier assumption of linear (C, \(\phi\)) characteristics (Barton, 1976).

The strength components \(\phi_b\) and \(i\) appearing in equation 4.1 are usually termed the "basic friction angle" and the "effective roughness" or "\(i\) value" for the case of jointed rock. Unfortunately the geometrical component \(i\) for a given joint surface is nearly impossible to estimate without performing a shear test. Patton, (1966) suggested that only the first-order irregularities would contribute to the shear strength of joints beneath natural slopes, since slope creep and weathering would probably cause failure of the smaller scale asperities. Barton (1976) has tabulated values of \(\phi_b\) obtained from shear testing of sand-blasted, rough-sawn surfaces (Table 4.1) and gives values for limestone of between 33 and 40°. The results of direct shear tests carried out by Huang et al, (1993) indicated that damage of asperity surfaces can also be important, particularly for cyclic sliding events.

Some authors (Patton, 1966 and Barton, 1973) have demonstrated that the effective friction angle of a clean rough joint may be considered as comprised of a basic friction angle (\(\phi_b\)) and an angle (\(i\)) which accounts for the contribution of surface roughness.

\(\phi_b\) has been described by Barton (1976) as the "value of arc tan (\(\tau/\sigma_n\)) obtained from residual shear tests on flat, unweathered rock surfaces which can conveniently be sand-
blasted between tests" Barton further suggests that $\phi_b$ is most closely simulated by artificial surfaces which show no stick-slip oscillations or increase in frictional resistance with displacement (surfaces too smooth) and no appreciable peak, nor fall to residual Stimpson, (1981).

The thickness of a joint is intuitively related to joint roughness and to the height of the surface asperities. If the asperity height is $\delta$, perhaps the joint thickness is $2\delta$. Under continued shearing, the asperity tips will fail and form a gouge that tends to fail the valleys between asperity peaks. As a consequence, the joint surface friction angle tends to decrease towards a residual value, (Parisau, 1992).

Since the shear normal failure curve may be nonlinear, it is important to use normals that represent the expected range of normals for potential failure geometries in the slope. The tests at each normal should be run with sufficient displacement to obtain a residual shear strength, as the residual strength usually is a better estimate of in-situ strength than the peak strength. Call, (1992) stated that the more general curve is the power with an intercept, that is

$$\tau = C + K \sigma_n^m$$ (4.3)

The linear is a special case of the power with the intercept where;

$m = 1$, in which case $K$ becomes $\tan$ and $C$ is cohesion.

For intact rock, unconfined compression and Brazilian disc tension test are recommended. In addition to obtaining the compression and tension strengths, the intact rock shear strength can be approximated by a fit to the tension and compression Mohair circles using the relationships,

$$\phi = \arcsin \left[ \frac{(U - T_0)}{(U + T)} \right] (0.85)$$ (4.4)

$$C = 0.5 T \tan \phi \left( \frac{1}{\sin \phi + 1} \right) (0.98)$$ (4.5)

where;
CHAPTER FOUR

The Shear Strength of Rock and Rock Joints

U is uniaxial compression and

T is tensile strength (τ)

The constants 0.85 and 0.98 are factors developed from comparison between triaxial testing and the simple linear fit to the uniaxial and disc tensile strength. For more stability analysis, the failure surface is not under high confinement, so triaxial testing is not necessary. Call, (1992). For the rock mass, where direct shear testing is not possible, indirect methods such as the rock mass rating (RMR) classification and back analysis must be used.

4.5 CONSTANT NORMAL STIFFNESS DIRECT SHEAR TESTING METHOD

According to the Bandis, (1990), there are two predominant approaches to the quantitative description of the mechanical properties of rock joints. The first one is theoretical approach and the second one is empirical approach. The theoretical approaches use various analytical concepts such as elasticity, plasticity, wear theory and contact theory in conjunction with established numerical modelling techniques (finite or distinct element methods). These methods usually required the determine the properties of complex material that cannot be obtained by the standard laboratory or field tests. In the other word, the properties are estimated from back-analyses of known results. This limits their use as predictive tools. In this regard, Haberfield and Johnston, (1994) believe that it is perhaps for these reason that the majority of practical rock engineering solutions involving rock joints are based on empirical approaches.

Empirical approaches deal with the analyses of field and laboratory test results to obtain correlation between joint and material characteristics and the observed shear behaviour. Consequently they tend to be site specific and they are not useable for the other sites and / or conditions. Most of the joint models have been developed for the constant normal load (CNL) condition and cannot readily applied to the more general constant normal stiffness (CNS) conditions, Haberfield and Johnston, (1994).
The results of a series of direct shear tests conducted on joint samples (concrete and soft argillaceous rocks) comprising regular and irregular triangular asperities under conditions of constant normal load (CNL) and constant normal stiffness (CNS) carried out by Kodikara and Johnston, (1994) showed that the joint behaviour which required prediction was extremely complex and was controlled by a number of variables, not least of which was the wide range of random irregular roughnesses that could be found in these joints. They concluded that the behaviour of joints is complicated by the inherent natural variability of the properties of the rock making up the joints. This progressive investigation commenced with an examination of planar rock - concrete joints which was extended to the triangular joints. The tests conducted under CNS conditions showed a significant work strengthening response after initial sliding took place with peak resistance corresponding to the development of shearing through and crushing of the asperities. The results showed that the principal difference between the responses of regular joints all failed at the same point whereas for the irregular points, the asperities failed at different shear displacements. As a consequence for irregular joints, it was possible to have asperities which were intact, sheared and inactive at any one moment.

For the tests conducted under CNL conditions, the peak shear resistance was recorded when sliding first took place and therefore the work strengthening characteristics of CNS test were noticeably absent. In the other word, it appeared that the mechanisms of failure were very similar to those reported for the CNS tests, (Kodikara and Johnston, 1994).

In these tests the principal influence of roughness variations was that while the regular joints showed a relatively brittle response with a high shear resistance at a small shear displacement, the irregular asperities were more ductile with a generally lower peak resistance, which in the case of CNS conditions, occurred at a greater shear displacement, (Kodikara and Johnston, 1994).

Based on the results achieved by Johnston and Lam, (1989) from the direct shear tests carried out on the concrete / rock joint samples, when a bored pile is formed in a soft or weak rock, the contact between the concrete of the pile and the wall of the rock socked is equivalent to a rough rock joint. and when the pile is loaded vertically, the side shear
resistance develops as a function of the shear characteristics of the rough interface between the concrete and the rock. It should be noted that it has been assumed that bonding between the concrete and the rock is not developed when the concrete is cast.

The relationship between shear and normal force is purely frictional and is independent of any forms of cohesion, (Patton, 1966). It should be noted that for unbounded concrete rock surfaces, such a relationship should seem quite reasonable but, based on a large number of laboratory tested samples carried out by Lam and Johnston, (1989). This independence from cohesion was established to be confirm for the planar interfaces with \(i = 0\). In the case of \(i > 0\) it was identified that some significant cohesive component was present.

The direct shear tests carried out by ladanyi and Archambault, (1970), and Barton and Bandis, (1980) on real jointed samples indicated that, the behaviour of real joints is likely to be strongly influenced by the irregular nature of roughness, (Kodikara and Johnston, 1994)

4.6 FACTORS AFFECTING THE SHEAR STRENGTH OF ROCK DISCOTINUITIES

4.6.1 Transition from Dilation to Shearing

Dilation plays an important role in discontinuity behaviour, particularly in the shear strength of joint surfaces. At the low normal stresses dilation behaviour dominates the deformation characteristics of irregular discontinuities, since the joint surface roughness remains intact during shearing. Lama and Vutukuri, (1978) stated that there are two ways of representation of dilation. The first and the most commonly used method of representation is the vertical displacement against the horizontal displacement. The second method of representation is the relationship between the vertical displacement with respect to the horizontal displacement \((du/dh)\), where \(du\) is the vertical displacement perpendicular to the direction of shear force, and \(dh\) is the horizontal displacement in the direction of application of shear force against a certain dimensionless
ratio such as \((\tau / \sigma_n)\) or \((\sigma_n / \sigma_c)\) are plotted. Patton (1966) obtained the bilinear maximum strength envelope shown in Figure 4.2.

\[ \phi = (\phi + i) \]  
(4.6)

Thus the irregularities (roughness) appear to provide an effective \((i)\) value but, by increasing the normal load, the mode of failure will change resulting in some of the irregularities on the joint surface probably being sheared off. The residual shear strength, in this condition, of the natural joint can be quantified by the following equation;

\[ \tau_p = C + \sigma_n \tan \phi_r \]  
(4.7)
Newland and Allely (1957) statistically evaluated the shear strength of a rock mass by assimilating the overall sliding surface to a plane containing interlocked saw-tooth irregularities and, from this, developed the following equation:

\[ \tau_p = C + \sigma_n \tan(\phi_u + i) \quad (4.8) \]

Where
\( \tau_p \) is the shearing stress
\( \phi_u \) is angle of friction sliding resistance along the contact surfaces of the teeth
\( i \) is the angle of inclination of the teeth with respect to the general sliding surface
\( \sigma_n \) is conventional normal stress.
Table 4.1 Experimental values of basic friction angle for various rocks by different authors (After Barton, 1976)  * 1 MN/ m² = 10 kg/cm²

<table>
<thead>
<tr>
<th>Rock types</th>
<th>Moisture condition</th>
<th>( \sigma_a ) (MN/m²)*</th>
<th>( \phi_b^{\circ} )</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amphibolite</td>
<td>dry</td>
<td>0.1 - 4.2</td>
<td>32</td>
<td>Wallace, (1970)</td>
</tr>
<tr>
<td>Basalt</td>
<td>dry</td>
<td>0.1 - 8.5</td>
<td>35 - 38</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0.1 - 7.9</td>
<td>31 - 36</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td>Conglomerate</td>
<td>dry</td>
<td>0.3 - 3.4</td>
<td>35</td>
<td>Kersmanovic, (1967)</td>
</tr>
<tr>
<td>Chalk</td>
<td>wet</td>
<td>0 - 0.4</td>
<td>30</td>
<td>Hutchinson, (1971)</td>
</tr>
<tr>
<td>Dolomite</td>
<td>dry</td>
<td>0.1 - 7.2</td>
<td>31 - 37</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0.1 - 7.2</td>
<td>27 - 35</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td>Gneiss(schistose)</td>
<td>dry</td>
<td>0.1 - 8.1</td>
<td>26 - 29</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0.1 - 7.2</td>
<td>27 - 35</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td>Granite (c. . g.)</td>
<td>dry</td>
<td>0.1 - 8.1</td>
<td>26 - 29</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0.1 - 7.9</td>
<td>23 - 26</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td>Limestone</td>
<td>dry</td>
<td>0 - 0.5</td>
<td>33 - 39</td>
<td>Patton, (1964)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0 - 0.5</td>
<td>33 - 36</td>
<td>Patton, (1964)</td>
</tr>
<tr>
<td></td>
<td>dry</td>
<td>0.1 - 7.1</td>
<td>37 - 40</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0.1 - 7.1</td>
<td>35 - 38</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td></td>
<td>dry</td>
<td>0.1 - 8.3</td>
<td>37 - 39</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0.1 - 8.3</td>
<td>35</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td>Porphyry</td>
<td>dry</td>
<td>0 - 1.0</td>
<td>31</td>
<td>Barton, (1971)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>4.1 - 13.3</td>
<td>31</td>
<td>Barton, (1971)</td>
</tr>
<tr>
<td>Sandstone</td>
<td>dry</td>
<td>0 - 0.5</td>
<td>26 - 35</td>
<td>Patton, (1964)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0 - 0.5</td>
<td>25 - 33</td>
<td>Patton, (1964)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0 - 0.3</td>
<td>29</td>
<td>Ripley and Lee, (1962)</td>
</tr>
<tr>
<td></td>
<td>dry</td>
<td>0.3 - 3.0</td>
<td>31 - 33</td>
<td>Kersmanovic, (1967)</td>
</tr>
<tr>
<td></td>
<td>dry</td>
<td>0.1 - 7.0</td>
<td>32 - 34</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0.1 - 7.3</td>
<td>31 - 34</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td>Shale</td>
<td>wet</td>
<td>0 - 0.3</td>
<td>27</td>
<td>Ripley and Lee, (1962)</td>
</tr>
<tr>
<td>Siltstone</td>
<td>wet</td>
<td>0 - 0.3</td>
<td>31</td>
<td>Ripley and Lee, 1962</td>
</tr>
<tr>
<td></td>
<td>dry</td>
<td>0.1 - 7.5</td>
<td>31 - 33</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td></td>
<td>dry</td>
<td>0.1 - 7.5</td>
<td>31 - 33</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td></td>
<td>wet</td>
<td>0.1 - 7.2</td>
<td>27 - 33</td>
<td>Coulson, (1971)</td>
</tr>
<tr>
<td>Slate</td>
<td>dry</td>
<td>0 - 1.1</td>
<td>25 - 30</td>
<td>Barton, (1971)</td>
</tr>
</tbody>
</table>
Table 4.1 (continued) Experimental values of basic friction angle for various rocks

<table>
<thead>
<tr>
<th>Rock type</th>
<th>moisture condition</th>
<th>$\sigma_n$ (MN/m$^2$)</th>
<th>$\phi_b^o$</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone</td>
<td>dry</td>
<td>0.1 - 2</td>
<td>30 - 32</td>
<td>El - Mherig, (1986)</td>
</tr>
<tr>
<td>Basalt</td>
<td>dry</td>
<td>0.0 - 2</td>
<td>26</td>
<td>El - Mherig, (1986)</td>
</tr>
<tr>
<td>Porphyry</td>
<td>dry</td>
<td>0.0 - 2</td>
<td>26 - 29</td>
<td>El - Mherig, (1986)</td>
</tr>
<tr>
<td>Granite</td>
<td>dry</td>
<td>0.1 - 2</td>
<td>25 - 29</td>
<td>El - Mherig, 1986</td>
</tr>
</tbody>
</table>

This relationship provides a basic framework for describing the results of field and laboratory tests on jointed rock. Its application relies on the evaluation of $\mu$ values. Patton's model of joint deformation behaviour (1966) simplifies the joint behaviour and simulates the conditions of the normal load range over which dilation is very limited. This is a basic method for interpreting the results of field and laboratory tests on jointed rock. However, the applicability of this concept relies on the evaluation of $\mu$ values. It should be noted that in the case of high values of $\mu$ the most easily sheared points will be removed, because they have the narrowest base.

The transition normal load may be quantified by Patton's equation

$$\sigma_t = \frac{C_o}{\tan(\phi_u + i) - \tan(\phi_o)}$$

(4.9)

Where

$i = \text{Angle of inclination of asperities' teeth}$

$\sigma_t = \text{Transition normal stress}$

$C_o$ and $\phi_o$ represent the Coulomb shear parameters of the solid rock material. The model proposed by Patton (1966), to describe the failure envelope of joints, is presented in Figure 4.2.
It should be noted that when the normal stress is equal to the uniaxial compressive strength of intact rock, dilation will be zero because, in this case, the necessary force for running the two plates is more than the necessary force required for failure of the intact rock. Experimental evidence reported by Patton (1966) shows that the bi-linear failure envelope describes the shear strength for shearing along plane surfaces containing a number of regular and equal teeth fairly well (Ladanyi and Archambault, 1970).

Figure 4.3 shows a set of shear strength curves for three types of failure in a heavily jointed rock mass. As the normal stress increases, the curve for the rock mass is lower than that for the intact rock as a result of the weakening effect of the close jointing. The Ladanyi and Archambault approach is attractive, because it involves a consideration of the mechanics of block movement and failure within a rock mass. However, it is difficult to apply in practice because of the choice of the various parameters \( h, n, s_c, K, L, f, \) and \( i \) which are required to solve the equations (Hoek and Bray, 1981).

![Figure 4.3](image)

**Figure 4.3** Representation of shear strength curves for three types of failure in heavily jointed, interlocking rock masses.
4.6.2 Increased Shear Strength due to Dilation

The increase of shear strength due to dilation has been studied by many authors (Lam and Johnston, 1989; Johnston et al, 1987; Ladanyi and Archambault, 1970). Dilatency in brittle rocks will be accommodated without restriction during shear under constant normal stress, while under constant or variable normal stiffness, dilation is partially or completely inhibited by the surrounding rock mass and the normal stresses acting on the fault zone increase with shear displacement (Archambault et al, 1992). The increase in the value of shear strength due to dilation can be calculated by the following equation:

\[ \tau_D = \left[ s_v + t_v \tan(\phi_f) \right] (1-a_s) \]  \hspace{1cm} (4.10)

Where

- \( V \) is dilation rate
- \( a_s \) is shear area ratio
- \( \phi_f \) is average value of friction angle when sliding occurs along irregularities with different orientations.

Because of some difficulties which derive from measuring \( a_s \) from the tested samples, the calculation of \( \tau_D \) becomes impossible. Ladanyi and Archambault, (1970) tried to measure the amount of \( a_s \) by the following method:

It is reasonable to assume that \( V/(1-a_s) \) is equal to a constant \( K \) for any value of \( \sigma \). Consequently \( \tau_D \), can be given by the following equation:

\[ \tau_D = \left[ \sigma + \tau \tan(\phi_f) \right] \]  \hspace{1cm} (4.11)

In this equation the necessary parameters for computation of \( \tau_D \) can be obtained, by making the approximations of \( \phi_f = \phi_u \) (angle of friction the planer surface of asperities teeth), \( \phi_u = \phi_0 \) and \( K = V \) (when \( a_s = 0 \) and \( \sigma = 0 \)).

The transition from dilation to shearing was observed by Ladanyi and Archambault (1970) in a study of shear behaviour of a jointed rock mass. They introduced the following equation for measuring the peak shear strength.
CHAPTER FOUR

The Shear Strength of Rock and Rock Joints

\[ \tau_p = \frac{\sigma_n (1-a_s) (\nu^o + \tan \phi) + a_s \tau_r}{1 - (1-a_s) \nu^o \tan \phi} \quad (4.12) \]

Where

- \( a_s \) is the proportion of the discontinuity surface which is sheared through projections of rock material.
- \( \nu^o \) is the dilation rate \( dy/dx \) at peak shear strength.
- \( \tau_r \) is the shear strength of the rock materials.

They concluded that as \( \nu^o \) increases with \( \sigma_n \) approximately linearly at the beginning and attain the value of unity, on the other hand it will decrease rapidly at low normal stresses, and will attain zero value at a very low rate. The dilation rate \( \nu^o \) can be calculated from a shear test. Ladanyi and Archambault (1970), after carrying out a large number of shear tests on rough surfaces, proposed the following equations:

\[ \nu^o = (1-\frac{\sigma}{\sigma_t})^k \tan i \quad (4.13) \]

\[ a_s = 1 - (1-\frac{\sigma}{\sigma_t})^k \quad (4.14) \]

Hoek and Bray (1981), adapted the following equation for determination of peak shear strength for a heavily jointed rock mass.

\[ \tau_p = \frac{\sigma (1-a_s) (\nu^o + \tan \phi) + a_s \eta \sigma_c \sqrt{\frac{1+n-1}{n} \left(1+n-\frac{\sigma}{\eta \sigma_c}\right)^2}}{1 - (1-a_s) \nu^o \tan \phi} \quad (4.15) \]

Where

- \( \sigma \) is the normal stress.
- \( \eta \) is a parameter representing the shear strength of the rock.
- \( \sigma_c \) is a parameter representing the cohesion of the rock.
- \( \nu^o \) is the dilation rate.
- \( \phi \) is the friction angle of the rock.
- \( a_s \) is the proportion of the discontinuity surface which is sheared through projections of rock material.
\[ V^* = (1 - \frac{\sigma}{\sigma_c})^k \tan \theta \]

(4.16)

and

\[ a = 1 - (1 - \frac{\sigma}{\sigma_c})^L \]

(4.17)

\( \sigma_c \) is the uniaxial compressive strength of individual block within the rock mass, and \( \eta \) is the degree of interlocking, which defines the freedom of the block to translate and rotate before failure.

The consideration of the shear strength of heavily jointed rock masses has been recognised as a vital engineering problem and a number of authors have published some excellent papers on this subject. Modelling was carried out in large numbers by Ladanyi and Archambault (1970). In their research each model contained 1800 blocks in 1.72 x 1.72 x 625 cm packed tightly together to form a 625 cm thick model slab. Biaxial loads were applied in different directions and three distinct types of failure occurred in relationship to the joint orientation.

- Shear along a well defined plane inclined to both discontinuity sets
- Formation of a narrow failure zone in which block rotation occurred in addition to the sliding and material failure of case 1.
- Formation of a kink-band of rotated and separated columns of 3, 4, 5, blocks.

The suggested values for \( K, L, \eta \) for the three types of failure are as follows:

Case 1 = Shear plane formation

\[ K = 4, \quad L = 1.5, \quad \eta = 0.5 \]

For a single rough discontinuity surface with \( \eta = 0.6 \) to allow for a loosening of the rock mass as a result of close jointing.
Case 2 = Shear zone formation;

\[ K = 5 \text{ to allow for increased freedom of the block to rotate, } L = \tan (i) \text{ and } \eta = 0.6 \]

which allows for a looser rock mass than in case 1.

Case 3 = Kink-band formation

\[ K = 5, \quad L = \left( \frac{2}{n_r} \right)^{3 \tan i} \quad \eta = 0.6 \]

and \( n_r \) is the number of rows of pieces in the kink band, and \( n_r \) usually is between 3 to 5, \( \eta = 0.5 \) to allow for the very loose condition of the rock mass.

Note that the values of \( L, K, \eta \) have been determined by Ladanyi and Archambault (1976) and vary for different positions of failure.

Typical results of strength prediction for irregular rock surfaces obtained by the proposed simulation model by Ladanyi and Archambault (1970) for \( i = 30^\circ \) are shown in Figure 4.4.

Figure 4.4 represents the maximum strength envelope corresponding to the tests carried out by Ladanyi and Archambault (1970) in different normal tests. In addition, variations of other essential parameters such as and \( \nu^0 \) are represented in this figure.

### 4.6.3 Joint Compressive Strength (JCS)

The strength and deformability of the rock mass is largely controlled by the weathered layer of rock on the surface of slopes. This is very important when the joint compressive strength is a small fraction of the compressive strength of the intact rock. The joint wall compressive strength (JCS) is equal to the unconfined compressive strength (\( \sigma_c \)) of the rock if the joint is unweathered. However, it may be reduced to approximately \( 1/4 \sigma_c \) if the joint walls are weathered.
Figure 4.4 representation of basic assumptions and expected results for the proposed failure model (After Ladanyi and Archambault, 1970)

A = Definition of the dilation rate $\dot{V}^o$ and the shear area ratio as

B = Expected failure envelopes for irregular rock surfaces and rock mass

C = Anticipated variation of $\dot{V}^o$ and as with normal pressure ($\sigma_n$).
There are a number of different methods for measuring the joint compressive strength (JCS) in weathered joints, such as uniaxial compressive strength, and point load tests on core samples of jointed rock. The best and most economical means of measuring (JCS) is using the Schmidt Hammer.

The Schmidt Hammer’s operation is based on the rebound of a spring loaded plunger after it impacts with the joint surface. The height of the rebound is measured on a scale and the reading is taken as a measurement of rock hardness. The Schmidt Hammer is an ideal and inexpensive tool for field work.

There are four types of Schmidt Hammer which are available, each with a different impact energy. Energy is stored in a spring which automatically releases at a prescribed energy level and impacts a mass against the plunger. The Different types of Schmidt hammer are as follows:

- M type, with 29.43 (N.m) impact energy
- N type, with 2.207 (N.m) impact energy
- L type, with 0.735 (N.m) impact energy
- P type, with 0.883 (N.m) impact energy

Barton & Choubey (1977) described how the JCS value can be measured in the field or in the laboratory using a Schmidt hammer. It should be noted that the first hammer was designed and calibrated for a horizontal impact direction, but now the calibration chart allows the instrument to be used successfully in different orientations. A N-type Schmidt hammer was used to measure the Joint Compressive Strength (JCS) of the joints. The results of the measured JCS’s for the tested samples are shown in Table 4.2.

According to the method explained, the compressive strength of the rock surface was calculated as being equal to 55.15 MPa for sections A and B and 36.1 Mpa for sections C and D from the Mugga II Quarry, the north and west parts of the Marulan quarry 15.17 and 14.02 respectively and also for the north and west parts of the Dunmore Quarry was measured at 34.47 and 38.61 respectively.
Table 4.2 Different values of Joint Compressive Strength (JCS) for different type of rocks from the quarries

<table>
<thead>
<tr>
<th>Type of rock</th>
<th>Joint compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porphyry, sections A and B</td>
<td>31.92 - 45.29</td>
</tr>
<tr>
<td>(Mugga II Quarry)</td>
<td>Mean (36.31)</td>
</tr>
<tr>
<td>Porphyry, sections C and D</td>
<td>45.57 - 62.74</td>
</tr>
<tr>
<td>(Mugga II Quarry)</td>
<td>Mean (44.76)</td>
</tr>
<tr>
<td>Limestone, Marulan Quarry</td>
<td>8.55 - 19.03</td>
</tr>
<tr>
<td>(Western section)</td>
<td>Mean (23.56)</td>
</tr>
<tr>
<td>Limestone, Marulan Quarry</td>
<td>9.86 - 20.48</td>
</tr>
<tr>
<td>(Northern section)</td>
<td>Mean (23.14)</td>
</tr>
<tr>
<td>Basalt, Dunmore Quarry</td>
<td>27.92 - 41.02</td>
</tr>
<tr>
<td>(North part)</td>
<td>Mean (36.83)</td>
</tr>
<tr>
<td>Basalt, Dunmore Quarry</td>
<td>31.85 - 45.36</td>
</tr>
<tr>
<td>(West part)</td>
<td>Mean (38.73)</td>
</tr>
</tbody>
</table>

4.6.4 The Effect of Surface Roughness on the Shear Strength of Jointed Rock

The joint roughness coefficient (JRC) indicates a sliding scale of roughness, which varies from approximately 0 to 20; from the smoothest to the roughest end of the spectrum.
The Joint roughness coefficient (JRC) is an accepted concept for the description of rock surface roughness (Carr and Warriner, 1987). Rengers (1971) developed a very useful method of measuring and recording the surface roughness of joints, though most investigations confine themselves to the measurement of roughness by using a profilograph or stereomicroscopy, depending upon the size of the surface (cited by Lama and Vutukuri, 1978). An empirical law based on laboratory shear tests has been used to calculate JRC (Barton & Choubey, 1977).

\[
JRC = \frac{\arctan\left(\frac{\tau}{\sigma_n}\right) - \phi_b}{\log_{10}\left(\frac{JCS}{\sigma_n}\right)}
\]  

(4.18)

Where

\( \tau \) is shear stress, \( \sigma_n \) is effective normal stress, \( \phi_b \) is the basic friction angle and JCS is the joint wall compression strength.

Figure 4.5 shows the corresponding range of JRC values associated with each roughness property (After Barton, 1977). Barton et al, (1974) suggested that the peak shear strength of joints \( \tau \) in rock could be represented by the following equation.

\[
\tau = \sigma \tan \left( \psi + JRC \log \frac{\sigma_j}{\sigma} \right)
\]  

(4.19)

Where

\( \tau \) = Peak shear strength

JRC is joint roughness coefficient

\( \sigma_j \) is joint compressive strength

\( \phi_r \) is residual friction angle

\( \sigma_n \) is effective normal stress

Barton's equation is plotted in Figure 4.6 for different values of JRC (20, 10, 5). The slope face roughness of different quarries has been considered in two ways. The first one was large scale and, at this stage, the roughness was considered in the joint survey stages. It was found that the joint roughness including different categories such as smooth, rough, or undulating. The second method for measuring the roughness of joints involved the use of a contour gauge as is shown in Figure 4.7. The measured roughness values for each of the samples tested are listed in Table 4.3.
Figure 4.5  Typical profiles of roughness in associated JRC values (After Barton, 1977).
Barton's equation not to be applied in this area

\[ \text{Ladanyi and Archambault equation} \]

\[ JRC = 20 \]

\[ JRC = 10 \]

\[ JRC = 5 \]

Residual strength of smooth rock surface

Effective normal stress \( \sigma_n \)
Joint compressive strength \( \sigma_J \)

\( \tau \)

\( \phi = 30^\circ \)

\( i = 20^\circ \)

Figure 4.6 Representation of Barton's prediction for the roughness shear strength of discontinuities.

Figure 4.7 Illustration of contour gauge used for measuring the discontinuity roughness.
Table 4.3 Roughness values for the different sections of quarries

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Joint Roughness Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porphyry, sections A and B</td>
<td>Measured</td>
</tr>
<tr>
<td>(Mugga II Quarry)</td>
<td>10 - 12</td>
</tr>
<tr>
<td>Porphyry, sections C &amp; D</td>
<td></td>
</tr>
<tr>
<td>(Mugga II Quarry)</td>
<td>12 - 14</td>
</tr>
<tr>
<td>Basalt, west part</td>
<td></td>
</tr>
<tr>
<td>(Dunmore Quarry)</td>
<td>10 - 12</td>
</tr>
<tr>
<td>Basalt, north part</td>
<td></td>
</tr>
<tr>
<td>(Dunmore Quarry)</td>
<td>10 - 12</td>
</tr>
<tr>
<td>Limestone, west part</td>
<td></td>
</tr>
<tr>
<td>(Marulan Quarry)</td>
<td>14 - 16</td>
</tr>
<tr>
<td>Limestone, north part</td>
<td></td>
</tr>
<tr>
<td>(Marulan Quarry)</td>
<td>14 - 16</td>
</tr>
</tbody>
</table>

4.7 MECHANICAL PROPERTIES OF ROCK AND ROCK MASS

In order to obtain the shear strength characteristics for use in slope design, some form of testing is usually required. This may be a very sophisticated laboratory or in-situ test depending on the nature of the problem being investigated. For important slopes, large scale in-situ tests for obtaining reliable shear strength values for critical discontinuities may be required (Ozgenglu, 1990).

Figure 4.8 Relationship between shear strength-versus normal strength for typical rock surface.
Shear tests carried out on smooth, irregular, clean discontinuity surfaces and intact rocks at a constant normal stress generally give a shear stress-normal stress curve, as shown in Figure 4.8. When a number of such tests are carried out at different ranges of the effective normal stresses a linear shear strength envelope will be obtained. The sliding of rock blocks along the discontinuity surface is resisted by the shear strength of the rock surfaces. Therefore, for any stability analysis, an essential parameter is consideration of the shear strength of the surfaces on which movement is likely to occur. From Figure 4.8 it can be seen that the shear strength of intact rock at zero normal stress ($\sigma_n$) is approximately equal to twice its tensile strength.

![Figure 4.9 Relationship between shear stress-versus effective normal stress](image)

The shear strength of smooth, clean discontinuities can be described by Coulomb's law:

$$\tau = \sigma'_n \tan \phi'$$  \hspace{1cm} (4.20)

Where

- $\phi'$ is the effective friction angle of discontinuity surface
- $\sigma'_n$ is the effective normal stress

In the case of the Mugga II Porphyry rock, as is shown in Figures 4.10 to 4.13, the angle $\phi'$ was found to be around $33.24^\circ$ for sections A and B, and was also found to be $35^\circ$ for sections C and D.
Figure 4.10 Peak Shear Strength Envelope for Natural Joint samples of Porphyry rock (Mugga II Quarry sections A and B)

\[ y = 0.656x + 2.003 \quad r^2 = 1.000 \]
Figure 4.11 Peak Shear Strength Envelope for Natural Joint Samples of Porphyry Rock (Mugga II Quarry Sections C and D)

\[ y = 0.725x + 4.270 \quad r^2 = 0.951 \]
Figure 4.12 Peak Shear Strength Envelope for Natural Joint Samples of Limestone (Marulan Quarry)

\[ y = 0.342x + 1.912 \quad r^2 = 0.979 \]

Normal Stress, MPa

Shear Stress, MPa

0 1 2 3 4 5 6
0 1 2 3 4 5 6
Figure 4.13 Peak Shear Strength Envelope for Natural Joint Sample of Basalt (Dunmore Quarry)

Shear Stress, MPa

Normal Stress, MPa

\[ y = 0.493x + 2.025 \]
\[ r^2 = 0.967 \]
The jointed limestone samples from Marulan Quarry was measured at a 18.88° internal friction angle and a 26.24° internal friction angle was found for the basalt jointed samples from the Dunmore Quarry.

When a discontinuity plane exists along the surface to be sheared, the tensile strength and cohesive strength on this surface are zero and the resistance to sliding will be generated by the frictional force only. In the case of an irregular surface both frictional forces and the interlocking of surface asperities have a considerable influence on the shear strength of discontinuities on the surface planes before sliding can occur.

The peak strength at constant normal stress is reached after a small shear displacement. With further displacement, the shear resistance falls until the residual strength is eventually reached (Brady and Brown, 1993). Figure 4.14 shows different forms of the direct shear test. Goodman (1976) pointed out that although this test may reproduce discontinuity behaviour adequately in the case of sliding of an unconstrained block of rock from a slope, it may not be suited for determination of the stress-displacement behaviour of discontinuities isolating a block that may potentially slide and fall from the periphery of an underground excavation. (Brady and Brown, 1993).

![Figure 4.14 Representation of different modes of direct shear test](image-url)
When a rough discontinuity sample is sheared with normal stress applied, dilatancy will occur as shown in Figure 4.15.

According to this figure measurement of the shear displacement and normal displacement is possible from the following equations respectively:

\[ \Delta U = \Delta Z \cos \psi + \Delta X \sin \psi \]  
\[ \Delta V = \Delta Z \sin \psi - \Delta X \cos \psi \]

If the shear resistance is assumed to be solely due to friction, the shear stress will be zero throughout. As the normal stress is increased, the value of dilatancy will decrease, because a greater proportion of the asperities will be damaged during shearing (Brady and Brown, 1993).

Figure 4.15 Illustration of dilatancy derived from the sample sheared by normal stress applied.
4.8 THE SCALE EFFECT ON THE SHEAR BEHAVIOUR OF ROCK JOINTS

The choice of an appropriate joint test-size during a shear strength investigation is generally based on economic and technical considerations. The high cost of large scale conventional shear tests often leads to the relatively cheaper alternative of laboratory testing of a small joint sample (Bandis and Barton, 1981).

The scale effects of discontinuities on shear strength have been studied by many authors. Pratt et al (1974) suggested that the reduction in peak shear strength was due to a decreasing actual contact area with an increasing joint size. It was assumed that, "there would probably be no scale effect if the contact area of small and large joints was the same" and that such might be the case for unweathered, perfectly mating joints under high normal stress.

Barton (1976) described the same results on the basis of a scale effect on the joint compressive strength (JCS) operating on different sized samples. Barton and Choubey (1977) suggested that the joint roughness coefficient (JRC) presents another potential source of scale effect on shear strength. Back analysis of their tilt tests showed that the JRC value of the 450 mm joint increased from 5.5 to 8.7 after the joint had been divided into smaller blocks. They have shown that JRC can be considered as a constant irrespective of the level of normal stress within the range of engineering interest.

Bandis and Barton (1981) suggested that the influence of scale effects on shear strength is poorly understood. Any significant improvement in understanding would require answers to the following questions:

- Are scale effects on shear behaviour an intrinsic characteristic of rock joints?
- What is the mechanism of shearing at different scales, and what are the factors controlling the magnitude of any scale effect?
- To what extent is individual joint behaviour relevant to the behaviour of rock masses?

Barton (1973) suggested that the peak shear strengths of joints, $\tau_p$, in rock could be represented by the following equation

$$\tau_p = \sigma_n \tan \left( JRC \log_{10} \left( \frac{JCS}{\sigma_n} \right) + \phi_r \right)$$  \hspace{1cm} (4.23)
Where

\[ \tau_p = \text{Peak shear strength} \]

\[ \sigma_n = \text{Effective normal stress} \]

\[ JRC = \text{Joint roughness coefficient} \]

\[ JCS = \text{Joint wall compressive strength (equal to} \sigma_c \text{ here)} \]

\[ \phi_r = \text{Residual angle of friction (equal to} \phi_b \text{ here)} \]

According to the equation 4 - 23 there are three components of shear strength.

- A basic frictional component given by \( \phi_r \)
- A geometrical component controlled by surface roughness (JRC)
- An asperity failure component controlled by the ratio \( JCS/\sigma_n \)

As Figure 4.16 shows, the asperity failure and geometrical component combine to give the net roughness component. The total frictional resistance is then given by \( (\phi_r = i) \).

Equation 4.20 and Figure 4.16 show that the shear strength of a rough joint is both scale dependent and stress dependent. As normal stress increases, the term \( \log_{10} (JCS/\sigma_n) \) decreases, and the net apparent friction angle decreases.

When the rock joint is unweathered, the joint wall compressive strength (JCS) is equal to the unconfined compressive strength and will be reduced approximately to \( 1/4 \sigma_c \) if the joint walls are weathered.

Bandis and Barton (1981) stated that if the joint compressive strength (JCS) is scale dependent, the envelopes in Figure 4.16 will imply that the scale effect on \( \tau_p \) would be maximum for joints of high JRC and minimum for joints of low JRC. Figure 4.5 shows different values of the joint roughness coefficient (JRC).

The physical appearance of Equation 4.23 for different values of the joint roughness coefficient (JRC) is shown in Figure 4.16. As shown in this figure the shear strength envelopes for rough, undulating joints (class A) are steeply inclined at low effective normal stress levels.
The shear stress ($\tau$) and shear displacement ($dh$) relationship represented in Figure 4.16 is a typical example of the overall scale effect on joint shear behaviour. Bandis et at, 1981 shows that increasing the block size or the length of the joint leads to:

- A gradual increase in the peak shear displacement $dh$
- An apparent transition from a "brittle" to "plastic" mode of shear failure
- A decrease of the peak dilation angle $d_n$
- Insignificant scale effects in the case of relatively planer and smooth joint types.

Figure 4.16 Classification of roughness and prediction of shear strength for non-planer rock joints. Each curve is numbered with the appropriate value of JCS in units of (MN/m²) [After Barton, 1976]

However, in view of the safety requirements of rock engineering structures, Barton, (1976) suggested that values of $\text{arc tan} \frac{\tau}{\sigma_n}$ larger than $70^\circ$ and any possible cohesion,
intercept should be discounted, hence the curvilinear envelopes in the left hand diagram of Figure 4.17.

As shown in Figure 4.16 the joint wall compressive strength (JCS) is a very important parameter for the shear strength of rough joints when stress levels are low, as is the case in most rock engineering problems. Since the compressive strength of the joint wall is an important factor in the shear strength of rough joints. It can be concluded that any process that causes a reduction in compressive strength should result in reduced shear strength. Also increasing some factors such as weathering, moisture content, and time of failure cause a reduction in the compressive strength of rocks, and may also cause a reduction in the peak shear strength.

Considering the results of triaxial shearing tests on artificial faults performed at stress levels of several hundred MN/m² indicates that there is an increasing error between prediction and test results if the effective normal stress (σₙ) exceeds the rock's unconfined compressive strength (σₑ). The measured shear strength is always appreciably higher than that predicted (Barton, 1976).

The strength of the asperities can be considered as the unconfined compressive strength. The increasing contact area presumably causes the compressive strength of the asperities to increase due to more effective confinement. According to Equation 4.23 if the joint compressive strength (JCS) value develops into the confined compression strength of the rock which is equal to the different stress (σ₁ - σ₃), then Equation 4.23 can apparently be generalised as follow

\[
\tau = \sigma_n \tan \left[ JRC \log_{10} \left( \frac{\sigma_1 - \sigma_3}{\sigma_n} \right) + \phi_b \right] \quad (4.24)
\]

Where

σ₁ is the axial stress at failure

σ₃ is the effective confining pressure

JRC is the joint roughness compressive strength

When σ₃ = 0, Equation 4.24 has the same form as Equation 4.23
Figure 4.17 Cumulative means shear stress - shear displacement (a) and vertical displacement versus shear displacement "dilation" (b) [After Bandis et al, 1981]
4.9 APPLICATION OF COULOMB’S CRITERION

The results of direct shear tests were examined using Coulomb’s Criterion. Shear strength ($\tau$) can be defined as a linear function of normal strength which can be calculated from the following equation:

$$\tau = C + \sigma_n \tan(\phi)$$

(4.25)

$C$ = Cohesive strength of joint  
$\phi$ = Friction angle

Parisau, (1992) suggested the following equation for resistance shear force $\tau$, when the applied force $N$ acts perpendicular to the sliding surface.

$$\tau = [\sigma_j \tan(\phi_j) + (A_j/A) + \sigma_r \tan(\phi_j) + K_j(A_r/A)]$$

(4.26)

where the subscripts $j$ and $r$ refer to joint and intact rock respectively. If the assumption that the rock and joint friction angles are equal and the joint is cohesionless, the composite strength is

$$\tau = \sigma \tan(\phi) + K_r(A_r/A)$$

(4.27)

The above equation shows that the cohesion of a jointed rock mass is likely to be only a small fraction of the cohesion of the intact rock. (Parisau, 1992).

The shear strength parameters obtained from the linear regression analysis of naturally jointed samples from different types of rock are given in Table 4.3. In addition, the results are also presented in Figures 4.10 to 4.13. It can be concluded that this method of analysis is very simple but is capable of defining the friction angle and cohesive strength of natural or saw cut rock samples in a shear stress versus normal stress curve.

4.10 APPLICATION OF POWER LAW FAILURE CRITERION

The results of direct shear tests were examined by the power law failure criterion and the shear strength parameters obtained from the naturally jointed samples are given in
Figures 4.18 to 4.25. In this method the relationship between shear strength ($\tau$) and normal strength ($\sigma_n$) is given by the following equation:

$$\tau = A \sigma_n^B$$ \hspace{1cm} (4.28)

where; $A$ and $B$ are constant parameters.

4.11 RESULTS AND DISCUSSION

In any rock mechanics project, samples should be tested and the results of experimental work should be analysed to obtain useful information from the site before final decisions are made. Direct shear strength tests on different naturally jointed rock samples within different ranges of normal stress from 2.5 to 15 MPa were carried out and showed a different peak shear strength due to the rock's properties.

A series of shear tests were carried out on different rock samples from the quarries. The shear strength parameters of jointed samples ($C_j$ and $\phi_j$) were determined by shear stress - displacement for Porphyry, Limestone, and Basalt respectively. From the tests undertaken, the cohesion strength of natural joints ($C_j$) and the frictional angle ($\phi_j$) of the different natural joints samples from the quarries were measured and the results are presented in Table 4.4. It should be noted that the results are analysed using the following techniques:

- Curve fitting using the linear regression analysis.
- Curvilinear regression analysis using power law theory.

From the tests results, the cohesion strength of the natural joints ($C_j$) of Mugga II Quarry samples from sections A and B were measured 2.03 MPa and 4.270 MPa for sections C and D of the quarry (porphyry) and 1.912 MPa for Marulan Quarry samples (limestone) and 2.02 MPa for the Dunmore Quarry (basalt) was established. The friction angle ($\phi_j$) from sections A and B was found 33.26° and also 35.94° for sections C and D, 18.88°, and 26.24° for limestone and basalt samples.
From the point of view of engineering geology the presence of some shear zones with a very high width, particularly in some parts of the Mugga II Quarry (south and eastern parts), can have a great influence on the instability of the benches in these parts of the quarries.

The results of direct shear tests and Peak shear strength envelopes of natural joint samples from the different type of rocks are given in Figures 4.18 to 4.25. In the weathered sample (sections A and B), the presence of soft materials such as clay minerals (due to weathering of the surfaces of the joint planes), can be considered responsible for decreases of internal friction in moderately weathered specimens as opposed to the samples from sections C and D which are not weathered.

Table 4.4 The Shear strength parameters for natural jointed samples from different quarries.

<table>
<thead>
<tr>
<th>Type of rock</th>
<th>Cohesion (Cj, MPa)</th>
<th>Friction angle (φj)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porphyry, sections A and B (Mugga II Quarry)</td>
<td>2.03</td>
<td>33.26</td>
</tr>
<tr>
<td>Porphyry, sections C and D (Mugga II Quarry)</td>
<td>4.270</td>
<td>35.94</td>
</tr>
<tr>
<td>Limestone, Marulan Quarry</td>
<td>1.912</td>
<td>18.88</td>
</tr>
<tr>
<td>Basalt, Dunmore Quarry</td>
<td>2.02</td>
<td>26.24</td>
</tr>
</tbody>
</table>

The effect of the shear strength of joints in the stability assessment of slope faces in open pit mines is very important. It should be noted that the effect of infilling materials should be considered in the stability assessment of the hard rock slopes. Due to this point, different types of infilling materials from different parts of the quarries were examined by the X-RD method. The results are presented in Appendix A. The types of infilling materials which were found included quartz, calcite and clay minerals along with Iron oxide.
Figure 4.18 Shear Stress - Shear Displacement Diagram for Natural Joints of Porphyry Rock from Sections A and B (Mugga II uarry)
Figure 4.19 Peak Shear Strength Envelope for Natural Joint Samples of Porphyry Rock from Sections C and D, (Mugga II Quarry)

\[ y = 0.725x + 4.272 \quad r^2 = 0.951 \]

\[ y = 3.481x^{0.525} \quad r^2 = 0.938 \]
Figure 4.20 Shear Stress - Shear Displacement Diagram for Natural Joints of Porphyry Rock Sections C and D (Mugga II Quarry)
Figure 4.21  Peak Shear Strength Envelope for Natural Joints of Porphyry Rock from Sections A and B (Mugga II quarry)

\[ y = 0.656x + 2.003 \quad r^2 = 1.000 \]
\[ y = 2.230x^{0.535} \quad r^2 = 0.955 \]
Figure 4.22 Shear Stress - Shear Displacement Diagram for Natural Joints of Limestone (Marulan Quarry)
Figure 4.23 Peak Shear Strength Envelope for Natural Joint Samples of Limestone (Marulan Quarry)
Figure 4.24 Shear Stress - Shear Displacement Diagram for Natural Joints of Basalt (Dunmore Quarry)
Figure 4.25 Peak Shear Strength for Natural Jopint Samples of Basalt (Dunmore Quarry)
4.12 CONCLUSION

This chapter describes the direct shear testing methods of natural joints and analysis of the recorded data from an experimental work on a series of shear tests. In this chapter the methods of direct shear testing and also the method of preparation of the samples as well as the apparatus used have been detailed.

Due to the results from the different type of rocks which are given in Figures 4.18 to 4.24, the following conclusions can be made:

- The peak shear strength increases with the increasing of shear displacement.
- The maximum value of shear strength is related to the first one mm displacement of the jointed sample.
- Because of the calibration of the gauges and the hydraulic pumps which are used in this test (minimum is 1 kN) it is not possible to carry out the direct shear test in low normal stress and, therefore, the accuracy of this test is dependent upon the calibration of the gauges. In addition, loading is manual so it is not possible to study the behaviour of the jointed sample in full detail.
- With the increases of normal stress, the peak shear strength increases to the maximum value and at this stage it will be constant with the increases of the displacement.
- For different type of rocks, all shear stress - displacement curves are similar in overall view, but the maximum shear strength and maximum displacement are different for different samples.
- The variation of shear strength in jointed samples is dependent upon the degree of weathering, type and also thickness of filling materials.
- Both linear and power law criteria are very close and parallel to each other. They both have a very high correlation coefficient with the experimental results, and both linear and power laws were found to be the simplest and closest model for the measuring of shear strength from jointed samples in the laboratory.
- The values of the shear strength of the actual rock joint samples, determined by laboratory testing, can be used for evaluating the stability of hard rock slopes, if the samples are representative of the rock mass.
Chapter 5
Assessment of Slope Stability in Surface Mining of a Porphyry Rock Mass
CHAPTER FIVE

ASSESSMENT OF SLOPE STABILITY IN SURFACE MINING OF A PORPHYRY ROCK MASS

5.1 INTRODUCTION

The stability of hard rock slopes is of utmost importance for safe, more viable and economic design of open pit mines. It is mainly controlled by the structural features such as faults, bedding planes, folds and also discontinuities present in the slope faces of rock masses. Stability analysis of slope faces requires laboratory assessment of the shear strength of discontinuities and the nature and extent of unstable blocks intersecting the quarry excavation. An assessment of the geotechnical parameters which influence the instability of slopes in a heavily jointed porphyry rock mass was carried out in the Mugga II Quarry. The method of investigation was based on a three dimensional joint survey of the slope faces and also laboratory assessment of intact rock samples and naturally jointed samples taken from the discontinuities.

The main aim of this chapter is to describe the means of obtaining the required parameters relation to the rock mass and structural features to aid the slope stability assessment for the surface mining of a porphyry rock mass. The secondary aim is statistical analysis of the physical and mechanical properties of discontinuities and also consideration of more significant factors influencing rock slope stability. Also presented is an application of a modified Rock Mass Rating system for evaluating in-situ rock mass strength in the Mugga II Quarry.
5.2 SITE INVESTIGATIONS AND GEOLOGICAL STUDY

The Canberra 1:100 000 Geological Sheet covers an area of about 2500 km\(^2\) in the southern Tablelands of New South Wales and the Australian Capital Territory. It forms the north central part of the Canberra 1:250 000 Sheet area (SI/55-16) and is bounded by latitudes 35° 00' S and 35° 30' S and longitudes 149° 00' E and 149° 30' E. About one-third of the Canberra sheet lies within the ACT and the rest is in New South Wales. Figure 5.1 shows a locality map of the area. This first edition of the Canberra 1:100 000 Geological Sheet is primarily designed to provide updated geological controls for resources exploration, urban development and education in the ACT and surrounding areas of New South Wales. Also the rationalisation of the local stratigraphy that it embodies will assist in preparation of a Third Edition of the Canberra 1:250 000 Geological Sheet and in a wider context contribute to our knowledge of the regional geology of the Lachlan Fold Belt (After Abell, 1991).

5.2.1 Geomorphology of Canberra

As mentioned above the Canberra Sheet area forms part of the Southern Tablelands of New South Wales. Relief is moderate, with physiographic features ranging from uplands and dissected tablelands to mature plains with wide valleys and perched alluvial basins. Undulating terrain in the north west with isolated hills and ridges at elevations of 550-650 m rises gradually south eastwards to rugged terrain reaching locally over 1000 m; the highest point is 1120 m at Mount Molonglo (RM 108/718). The general north west direction of the main drainage results from the position of the mapped area immediately west of the Main Divide. Most streams tend to either follow or cut sharply across the meridional tectonic grain of the underlying geology. The country is drained largely by the Molonglo and Yass drainage basins. Streams in these basins flow towards the Murrumbidgee River, which acts as a local base level. Lake George is the center of a local basin of internal drainage (Abell, 1991). Figure 5.2 shows a semi-diagrammatic geomorphological map of Canberra.
5.2.2 Mugga Mugga Porphyry Member

This unit was first described, but not formally named or defined, by Pittman (1911). He described it as an extensive "massif" of quartz porphyry typically developed in the mountain known as Mugga Mugga. The name is derived from Mount Mugga Mugga (813 m, MR 932/855).
Pitman (1911) interpreted quartz porphyry outcropping on Mount Mugga Mugga as intrusive into late Silurian strata but did not name it. Mahoney & Taylor, 1913 described the porphyry as an extensive dacitic tuff and placed it within a broadly based stratigraphic.

Figure 5.2 Illustration of geomorphology of Canberra region (After Abell, 1991).
unit termed the 'Mugga Series'. Opik, (1958) first named the unit as the "Mugga Mugga" porphyry and supported Pittman’s interpretation by describing the porphyry as an expansive stock. Figure 5.3 shows a stratigraphy column of the Canberra formation compiled by Abell, (1991) from drill hole data.

Figure 5.3 Stratigraphy of the Canberra formation (After Abell, 1991).
5.3 GEOLOGICAL INVESTIGATION OF MUGGA II QUARRY

A geological field investigation was carried in Mugga II Quarry in order to study the geology and geotechnical characteristics of the porphyry rock mass. Based on the field investigations, it was concluded that, the existing rock mass in Mugga II Quarry is a compact blue-gray quartz-feldspar porphyry. The eastern and southern parts of the quarry containing a semi weathered porphyry rock and the north and western parts consist of fresh porphyry rock. The main porphyry rock mass is cut by several shear zones mainly in eastern part of the quarry. As is presented in the stability analysis of the slope faces in section 5.8 the porphyry rock mass is cut by a few major joint sets in different parts. A general view of Mugga II Quarry is presented in Figure 5.4

5.4 STATISTICAL ANALYSIS OF DISCONTINUITY DATA FROM MUGGA II QUARRY

Statistical analysis methods are used to study quantitatively and qualitatively the variations in field and laboratory data. Statistical analysis of the data collected from field investigations also gives a wide range of information required to understand the behaviour of the rock and also to recognise the relationship and interaction between the rock mass characteristics, particularly with regards to slope stability in open pit mining. The intensity of fracturing in rocks is of fundamental importance in considerations of rock mass properties before, during and after excavation. For this reason, estimates of the discontinuity frequency are normally made during a site investigation for any major civil or mining engineering scheme, Priest and Hudson, (1981).

Since the inherent nature of rock masses is such that they are anisotropic and non uniform containing varied geological structures such as discontinuities in hard rocks, instability of the rock masses can clearly occur. Therefore, a statistical analysis of the field data can make a full understanding of the behaviour of the rock mass possible.
Figure 5.4 General view of Mugga II Quarry
Methods of statistical analysis enable researchers to compare the performance of the different testing methods and also allow comparison between the effects of the different types of discontinuities.

Although the characteristics of discontinuities were fully described in chapter two, a statistical study was carried out to consider the importance of each type of discontinuity and also their interactions particularly in relation to open pit mining. The most important statistical characteristics which are considered are central tendency, median, dispersion, variance, standard deviation, and relative frequency distribution (Appendix 1).

5.4.1 Descriptive statistical analysis

In order to analyse the discontinuity data, both descriptive statistics (e.g. frequency distribution) and inferential statistics (analysis of variance, and matrix correlation) were used. In descriptive statistical techniques, the discontinuity data obtained from the site investigations were classified as numerical data. When using frequency distribution as one of the techniques, a two-column table is used to summarise a set of data. One column shows the distinct score value in the data set and the other column shows the number of times that each score value occurs in the data set.

By using inferential statistical techniques, the purpose was to find inter correlations between different parameters (e.g. the relationship between variable parameters used as aperture, spacing, persistence, roughness, curvature, types of discontinuities and filling material). The proportions of bivariate relationships can become positive or negative and strong or weak.

Analysis of variance as an inferential statistical technique is performed to determine if there is any significant difference between different parameters in different sites (e.g. type of rock by different sites). To give a clearer picture of the effects of type of rock on discontinuity parameters.

In this chapter is presented the results from the statistical analysis of the relationships between discontinuity parameters that have been measured by joint survey programs, and discontinuity parameter differences regarding slope stability problems in different open
CHAPTER FIVE  Assessment of Slope Stability in Surface Mining of a Porphyry Rock Mass

pit mines. The relationships between discontinuity parameters in different parts of each open pit mine and also a statistical comparison between different types of rocks from the quarries were studied.

The discontinuity data referred to a wide range of information achieved from site investigations and geological mapping of the quarries which shows the characterisation of the discontinuities and their behaviour. It should be noted that in this statistical analysis specifications of the discontinuity parameters were recorded by using categories (for example the curvature parameter was recorded in three different categories so that number one was stepped, number two was undulating and number 3 showed planar form).

The results presented are based on data collected through joint surveying at the sites. In the field investigation discontinuity parameters were recorded and the results were used both in statistical analysis and a stereographic projection analysis method. Four parts of Mugga II Quarry were studied.

The initial phase of the data analysis shows the percentage distribution of different discontinuity parameters in Mugga II Quarry. Second and more importantly is the consideration of the significant differences between different parameters. It should be noted that all percentages are rounded, and figures are correct to two decimal places.

5.4.1.1 Orientation of discontinuities

The orientation of discontinuities has a great effect on the stability of the slope faces in open pit mining operations. The orientation of discontinuities can be directly measured by using a geological compass clinometer. Discontinuity orientation data can be presented and utilised by two different methods: if the rock exposures are easily accessible the orientation of discontinuities can be measured and employed in the design of slope face orientations. In the second method, if the rock face is not accessible it is necessary to use a statistical model using discontinuity orientation measured at other rock face that presents the discontinuity orientation characteristics of the rock mass. One of the simplest forms of graphical representation is the rose diagram, Attewell and Farmer, (1976). The disadvantage of rose diagram is that they contain no information on dip angle. This disadvantage can be overcome by selecting data from the more significant
class intervals and then plotting a histogram of dip angles, Priest, (1993). The difficulties of representation of three dimensional orientation data in two dimension can be overcome by using the stereographic or hemispherical projection techniques used by many authors including Goodman, (1976); Phillips, (1979); Warburton, (1980); Priest, (1985); Hoek and Bray, (1981); Brady and Brown, (1993). The frequency distribution of the discontinuity orientations in different parts of the quarry was examined using a statistical analysis method and results are given in the Table 5.1. It is clear from this table that of discontinuity orientations are varied in different parts of the quarry. Figures 5.5 to 5.8 show the rose diagram for section A to D. These rose diagrams show how the orientation and the relative significant of clusters of preferred dip direction are clearly visible.

Table 5.1 Results of Descriptive Statistical Analysis of Discontinuity Orientations Data.

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of Values</th>
<th>Mean</th>
<th>Std. Div (degree)</th>
<th>Minimum</th>
<th>Maximum concentration at ( % )</th>
<th>Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>270</td>
<td>218.69</td>
<td>70.62</td>
<td>80</td>
<td>240 - 260</td>
<td>21.11</td>
</tr>
<tr>
<td>Section B</td>
<td>363</td>
<td>235.9</td>
<td>67.19</td>
<td>70</td>
<td>100 - 120</td>
<td>17.72</td>
</tr>
<tr>
<td>Section C</td>
<td>282</td>
<td>201.36</td>
<td>32.31</td>
<td>60</td>
<td>300 - 320</td>
<td>15.34</td>
</tr>
<tr>
<td>Section D</td>
<td>201</td>
<td>188.58</td>
<td>76.37</td>
<td>20</td>
<td>120 - 140</td>
<td>22.39</td>
</tr>
</tbody>
</table>

5.4.1.2 Dip angle

The frequency distribution of dip angle was examined for the Mugga II Quarry and the statistical analysis results for sections A to D are given in Table 5.2. It is clear from Table 5.2 that these results indicate that the high value of the dip angle can influence the stability of the slope faces in different parts of this quarry. A graphical presentation of the frequency distribution of the dip angle in section A to D are given in Figures 5.9 to 5.12. According to the results of stability assessment derived from the stereographic projection these high values of dip angle have influenced on the instability of slope faces of the quarry and as it is described in section 5.6 some new dip angle have proposed as a remedial work in order to prevent the potential failure in different parts of the quarry.
Figure 5.5 Rose diagram of 270 joint traces in Section A

Figure 5.6 Rose diagram of 282 joints in Section B

Figure 5.7 Rose diagram of 363 joint traces in Section C

Figure 5.8 Rose diagram of 201 joints in Section D
Figure 5.9 Probability Density Histogram of the Plane Inclination In Section A

Figure 5.10 Probability Distribution of Discontinuity Dip Angle in Section B

Figure 5.11 Probability Distribution of Discontinuity Dip Angle in Section C

Figure 5.12 Relative Frequency of Discontinuity Dip Angle in Section D
### Table 5.2 Frequency distribution of discontinuity dip angle in different parts of quarry

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of Values</th>
<th>Mean</th>
<th>Std. Div</th>
<th>Minimum (degree)</th>
<th>Over 60 degrees (%)</th>
<th>Maximum concentration at</th>
<th>Mode (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>270</td>
<td>63.99</td>
<td>25.65</td>
<td>5</td>
<td>73.5</td>
<td>70 - 80</td>
<td>28.15</td>
</tr>
<tr>
<td>Section B</td>
<td>363</td>
<td>71.19</td>
<td>17.74</td>
<td>4</td>
<td>80</td>
<td>70 - 80</td>
<td>41.13</td>
</tr>
<tr>
<td>Section C</td>
<td>282</td>
<td>60.1</td>
<td>20.64</td>
<td>5</td>
<td>84.5</td>
<td>70 - 80</td>
<td>37.74</td>
</tr>
<tr>
<td>Section D</td>
<td>201</td>
<td>68.19</td>
<td>21.01</td>
<td>4</td>
<td>73</td>
<td>80 - 90</td>
<td>33.33</td>
</tr>
</tbody>
</table>

#### 5.4.1.3 Discontinuity aperture

The aperture of the discontinuity values is the perpendicular distance measured between discontinuity walls of joint plates which may be filled by different types of infilling materials. Results of statistical analyses in section A shows that in the quarry the highest percentages (38.50 and 32.96) of apertures are described by categories less than 2 mm and between 2 - 6 mm respectively. A graphical presentation of aperture frequency distribution in section A is given in Figure 5.13.

In the case of section B of the quarry, results of statistical analysis of discontinuity data shows that the highest percentages (56% and 21.6%) of aperture frequency distribution are found in categories four and five. Figure 5.14 shows a graphical presentation of frequency distribution in section B. It should be noted that in this section because of the influence of weathering more than 56% of the apertures belongs to the 6 mm category. Similar distributions have not been observed in other parts of the quarry.

In section C the highest percentages of discontinuity apertures 52.06% and 26.17% were found in categories 2 -6 and 6 - 20 mm respectively. Figure 5.15 represent a graphical presentation of discontinuity apertures in section C of the quarry. The results of statistical analysis of data in section D of the quarry indicated that the highest percentages of discontinuity apertures, 37.81% and 31.34% belong to categories 2 - 6 and 6 - 20 mm. From the point of view of engineering geology these high values for the apertures in this part of the quarry can be developed by the few faults which are present.
in this section of the quarry. Figure 5.16 shows a graphical presentation of discontinuity apertures in section D.

5.4.1.4 Discontinuity infilling materials

As described in chapter 4, the thickness and types of infilling material play an important role in the shear strength of the discontinuities. The influence of the infilling material in the discontinuity shear strength is such that when the thickness of the infilling material is greater than the amplitude of the asperities, the shear strength of the discontinuities will be governed by the shear strength of the infilling material.

In the section A of the quarry, it was found that the most important infilling material types are the clear and staining types, with 42.93% and 12.96% respectively. A graphical presentation of frequency distribution is given in Figure 5.17.

In section B, statistical analysis shows that more than 35% of the discontinuities are free of infilling materials (e.g. clear). This means that infilling material has probably been dissolved by rain fall. The results of statistical analysis of infilling material for section B are given in Figure 5.18.

Statistical analysis of data in part C of the quarry show that most parts of discontinuities are filled with cemented materials with the highest percentage being 26.17%. According to the results of field investigations and also inspection of the filled samples, this type of infilling material effects the shear resistance of the discontinuities and may cause an increase in shear strength of discontinuities in this part of quarry.

In section D of the quarry, most of discontinuities were clear with 38.31% and no infilling material except that in some cases they were filled with different types of infilling materials. It should be noted that in this part of the quarry traces of seepage and water flow were observed and it can be concluded that the infilling materials were probably dissolved by the water over time. Figures 5.19 and 5.20 show graphical presentation of frequency distribution of infilling materials in sections C and D respectively.
Figure 5.13 Frequency Distribution of Discontinuity Aperture in Section A

Figure 5.14 Probability Distribution of Discontinuity Aperture in Section B

Figure 5.15 Probability Density Histogram of Discontinuity Aperture in Section C

Figure 5.16 Probability Distribution of Discontinuity Dip Angle in Section D
Figure 5.17 Probability Density Histogram of Discontinuity Infilling in Section A

Figure 5.18 Frequency Distribution of Discontinuity Infilling in Section B

Figure 5.19 Relative Frequency of Discontinuity Infilling in Section C

Figure 5.20 Frequency Distribution of Discontinuity Infilling in Section D
5.4.1.5 Joint Compressive Strength

Using the Schmidt rebound hammer for measuring the joint compressive strength of rock is a very simple and quick method so that it can be used in the field to confirm the quality and approximate rock strength. The detail of method is given in section 4.5.3 of chapter four. In this chapter the results of measured JCS on the examined rock slope faces in different parts of quarry are given as probability density histograms in Figures 5.21 to 5.24. As it is clear from these figures, frequency distribution of joint compressive strength in different parts show a normal distribution form and the mean value of each histogram is as representative of the joint compressive strength (JCS) of each individual section. Figures 5.21 to 5.24 show graphical presentation of frequency distribution of joint compressive strength in sections A to D.

5.4.1.6 Discontinuity water condition

The pore water pressure is one of the most important parameters affecting instability of slope faces in open pit mining. With regard to this point the water condition in different parts of Mugga II Quarry were observed in field investigations. As described in chapter two the groundwater and pore water pressure is amongst the most important parameters in the instability of the slope faces in open pit mining operations. In section A, only 11% of the discontinuities showed traces of water flow or water seepage and 89% of displayed dry conditions.

In section B of the quarry water condition is classified dry with 98.93% and only 1.07% of discontinuities showed some traces of seepage. This means that this factor has not a great effect on the instability of slope faces. Section C is situated in the southern part of the quarry and from the results of the statistical analysis it was found that all (100%) of discontinuity water conditions belong to category one (e.g. dry condition). Therefore, this parameter has not any effect on the instability of the slope faces in this part of the quarry.

From the results of statistical analysis from section D it was found that more than 84% of discontinuities belong to category one (e.g. dry condition).
Figure 5.21 Frequency Distribution of JCS in Section A

Figure 5.22 Probability Density Histogram of JCS in Section B

Figure 5.23 Probability Density Histogram of JCS in Section C

Figure 5.24 Frequency Distribution of JCS in Section D
All together more than 15% of discontinuities have showed traces of seepage and water flow in this section of the quarry. This means that the pore water pressure is present in this part of the quarry and it should be considered as an instability parameter. Table 5.3 shows frequency distribution of water conditions in section A to D.

### Table 5.3 Statistical Analysis Results of Discontinuity Water Condition in sections A to D of Mugga II Quarry

<table>
<thead>
<tr>
<th>Categories</th>
<th>Section A</th>
<th></th>
<th>Section B</th>
<th></th>
<th>Section C</th>
<th></th>
<th>Section D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
<td>Number</td>
<td>Percent</td>
<td>Number</td>
<td>Percent</td>
<td>Number</td>
</tr>
<tr>
<td>Dry</td>
<td>242</td>
<td>89.63</td>
<td>279</td>
<td>98.93</td>
<td>363</td>
<td>100</td>
<td>169</td>
</tr>
<tr>
<td>Seepage</td>
<td>15</td>
<td>5.56</td>
<td>3</td>
<td>1.07</td>
<td>0.0</td>
<td>0.0</td>
<td>19</td>
</tr>
<tr>
<td>&lt;0.1 l/sec</td>
<td>13</td>
<td>4.81</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>13</td>
</tr>
<tr>
<td>Total values</td>
<td>270</td>
<td>100</td>
<td>363</td>
<td>100</td>
<td>282</td>
<td>100</td>
<td>201</td>
</tr>
</tbody>
</table>

#### 5.4.1.7 Discontinuity curvature

The curvature and waviness of the discontinuities in the slope faces of the Mugga II Quarry were qualitatively examined according to the ISRM (1981) suggestions. The results obtained from the statistical analysis showed that the curvature of the discontinuity slope faces in section A represented a high percentage (57.04%) of planar type in this part of the quarry, although 31.48% of discontinuities are belong to the category two (e.g undulating). This type of curvature can lead to acceleration of sliding in this part of the quarry. In section B of the quarry the curvature of the discontinuities with the highest percentage of discontinuities (75.89%) belongs to category three (e.g planar). In this part of quarry less than 25% of the discontinuities belong to the other categories. This means that the this type of curvature (planar) significantly affect the instability or sliding of the slope faces.

In section C the results of statistical analysis of data show that the curvature of the discontinuities within the slope faces, representing a highest percentage (61.70%) of discontinuities belong to the planar category. In this section less than 39% of discontinuities belong to the other categories.
Figure 5.25 Probability Density Histogram of Discontinuity Curvature in Section A

Figure 5.26 Frequency Distribution of Discontinuity Curvature in Section B

Figure 5.27 Frequency Distribution of Discontinuity Curvature in Section C

Figure 5.28 Probability Density Histogram of Discontinuity Curvature in Section D
Although, the maximum concentration of discontinuity curvatures are in the planar category but, around 40% of them belong to the stepped and undulating categories which affects the increased shear resistance of discontinuities in this part of the quarry. From the statistical analysis of discontinuity data in part D of quarry it was found that more than 50% of discontinuity curvatures were planar and around 42.2% belong to the undulating category. Therefore, in this particular section the planar type is dominant and has a great effect on the potential for failure or sliding of the rock blocks. Table 5.4 shows the frequency distribution of discontinuity curvature in different parts of quarry. In addition graphical presentation of the frequency distribution of discontinuity curvature are given in Figures 5.25 to 5.28

5.4.1.8 Discontinuity roughness

In order to assess the discontinuity roughness in different parts of the Mugga II Quarry a contour gauge was used (Figure 5.61). In this investigation small scale roughnesses of discontinuity surfaces were examined by comparison of each 10 cm. Length of the joint surfaces with the standard profiles suggested by ISRM (1981). The results of the statistical analysis of the large scale discontinuity roughness in section A of the quarry

Table 5.4 Frequency distribution of discontinuity curvature in different parts of quarry

<table>
<thead>
<tr>
<th>Categories</th>
<th>Section A</th>
<th>Section B</th>
<th>Section C</th>
<th>Section D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>stepped</td>
<td>31</td>
<td>11.48</td>
<td>36</td>
<td>12.76</td>
</tr>
<tr>
<td>Undulating</td>
<td>85</td>
<td>31.48</td>
<td>32</td>
<td>11.34</td>
</tr>
<tr>
<td>Planar</td>
<td>154</td>
<td>57.04</td>
<td>214</td>
<td>75.89</td>
</tr>
<tr>
<td>Total values</td>
<td>270</td>
<td>100</td>
<td>363</td>
<td>100</td>
</tr>
</tbody>
</table>

show that more than 55.18% of discontinuities belong to the first category (e.g. rough) which means that their influence is an increase of the shear strength parameters. Figure 5.29 illustrates the frequency distribution of discontinuity roughness in section A of the quarry.
Figure 5.29 Frequency Distribution of Discontinuity Roughnesses in Section A

Figure 5.30 Probability Density Histogram of Discontinuity Roughnesses in Section B

Figure 5.31 Probability Density Histogram of Discontinuity Roughnesses in Section C

Figure 5.32 Frequency Distribution of Discontinuity Roughnesses in Section D
Figure 5.30 shows that more than 62.5% of discontinuities belong to the second category (e.g. smooth). It can influence in decreasing of the shear strength parameters and it can lead to sliding of rock blocks in this part of quarry. Figure 5.30 illustrates the frequency distribution of discontinuity roughnesses in section B of Mugga II Quarry.

From the results of statistical analysis of discontinuity data in section C it was found that more than 60% of discontinuity roughness is classified in the rough category. This means that this parameter can influence increasing of the shear strength of discontinuities and can influence the stability of rock blocks. Figure 5.31 shows frequency distribution of discontinuity roughness in part C of quarry.

It is clear from Figure 5.32 that more than 50% of discontinuity roughness are in the smooth category since 47% of them belong to the rough category. In this part of the quarry there is nearly a balance between these two categories but the smooth category is dominant. This means that they affect the instability or sliding of the rock blocks in this part of the quarry. Figure 5.28 shows a graphical presentation of the frequency of discontinuity roughness in part D of Mugga II Quarry.

### 5.4.1.9 Discontinuity persistence

Discontinuity persistence is among the parameters which have the most significant influence on rock mass strength and it has a great influence on rock mass sliding resistance but there are some difficulties in relation to the direct mapping of discontinuity persistence into the rock mass (Figure 5.33). With reference to a joint plane (a plane through the rock mass that contains a patchwork of discontinuities and intact-rock region), joint persistence $K$ is defined by Einstein et al. (1983) as the fraction of area that is actually discontinuities and it can be expressed by $K$ as follows:

$$K = \lim_{A_D \to \infty} \frac{\sum a_{Di}}{A_D} \quad (5.1)$$

Where;

- $D$ is a region of the plane with area $A_D$ and
- $a_{Di}$ is the area of the ith joint in D (Figure 5.33).

- $a_{Di} = \text{Area of individual joint}$
- $A_D = \text{Area of joint plane}$
In other words, it can be said that discontinuity joint persistence (K) can be defined as:

$$K = \lim_{A_D \to \infty} \frac{\sum a_{Di}}{A_D} = \frac{\text{Jointed Area}}{\text{Total Area}}$$  \hspace{1cm} (5.2)

The summation in equation (5.1) is over all joints in D. Equivalently, joint persistence can be expressed as a limit length ratio along a given line on a joint plane. In this case,

$$K = \lim_{L_s \to \infty} \frac{\sum L_i}{L_s}$$  \hspace{1cm} (5.3)

Where

$L_s$ is the length of a straight line segment $S$ and $L_i$ is the length of the $i$th joint segment in $S$; or for a particular joint (Figure 5.34)

Figure 5.34 Illustration of joint persistence as length ratio. (After Einstein et al, 1983)
Joint persistence can be used to estimate the strength of a rock mass resisting sliding along a given plane; if the plane of sliding has area $A$, then shearing resistance in the case of intact rock can be expressed as:

$$R_r = (\sigma_a \tan \phi_r + C_r)A$$  \hspace{1cm} (5.5)

and this shearing resistance for the rock mass can be expressed by:

$$R_j = (\sigma_a \tan \phi_j + C_j)A$$  \hspace{1cm} (5.6)

It should be noted that in the case of a completely jointed region, $\phi_r$ and $\phi_j$ are the friction angles of intact rock and the joint respectively, $C_r$ and $C_j$ are the intact rock and joint cohesion.

ISRM, (1978) proposed the termination index ($T_i$) by the following equation:

$$T_i = \frac{100N_i}{N_i + N_a + N_o} \%$$  \hspace{1cm} (5.7)

Where

$N_i$, $N_a$, and $N_o$ are, respectively, the total number of discontinuities, whose semi-trace terminations are in intact rock, and at other discontinuities are obscured calculated for the complete scanline sample or for a specified discontinuity set. A large value of $T_i$ indicates that a large proportion of discontinuities terminate in intact rock, suggesting that the rock mass contains many intact bridges rather than being separated into discrete blocks. It might be expected therefore that a rock with a high termination index ($T_i$) would be relatively stiffer and stronger than a mass with a lower index, that it would not be susceptible to rigid block failures and would have a lower mass permeability Priest, (1993).

The persistence of the discontinuities was considered in the joint survey program and from the results of statistical analysis of discontinuity data it was found that most of the discontinuity terminations in section A belong to the category one (e.g. at another) with
74.44%. It can be confidently said that the persistence at this rate can influence the distribution of joint systems and consequently the instability of the slope faces.

In section B, from the results of statistical analysis of data it was found that discontinuity persistence is mainly governed by the first category (e.g. at another) with a high percentage of 74.11% which means that there is a net-work and continuous jointed system in this part of the quarry. This means that with this high percentage it can lead to instability of the slope faces in this part of the quarry.

From the results of statistical analysis of discontinuities data from section C it was found that more than 62.5% of discontinuities belong to the first category (e.g. at another). This means that the rock mass is very heavily jointed and this can cause to the decreasing of the quality and the strength of the rock mass in this particular part of the quarry.

In section D the discontinuity persistence is mainly governed by the first category (e.g. at another) with the high percentage of 74.13% which means that there is some continuous jointed systems in this part of the quarry. This higher percentage of discontinuity persistences can lead to the instability of the slope faces in this part of the quarry. The frequency distribution of discontinuity persistences are given in Table 5.5. Figures 5.35 to 5.38 illustrate the frequency distribution of the persistences in sections A to D.

Table 5.5  Frequency distribution of discontinuity persistences in Sections A to D.

<table>
<thead>
<tr>
<th>Categories</th>
<th>Section A</th>
<th>Section B</th>
<th>Section C</th>
<th>Section D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>At another</td>
<td>201</td>
<td>74.44</td>
<td>209</td>
<td>74.11</td>
</tr>
<tr>
<td>In rock</td>
<td>36</td>
<td>13.33</td>
<td>18</td>
<td>6.34</td>
</tr>
<tr>
<td>Beyond exposure</td>
<td>33</td>
<td>12.22</td>
<td>55</td>
<td>19.50</td>
</tr>
<tr>
<td>Total values</td>
<td>270</td>
<td>100</td>
<td>363</td>
<td>100</td>
</tr>
</tbody>
</table>
Figure 5.35 Frequency Distribution of Discontinuity Termination in Section A

Figure 5.36 Relative Frequency of Discontinuity Roughnesses in Section B

Figure 5.37 Probability Density Histogram of Discontinuity Termination in Section C

Figure 5.38 Frequency Distribution of Discontinuity Termination in Section D
5.4.1.10 Discontinuity spacing

It is clear that all rock masses contain discontinuities and it is not possible to find rocks that are completely intact. Discontinuities are present in rock masses and their presence strongly affects the mechanical and hydrogeological properties of rock masses, particularly in terms of strength and stability (Sen et al, 1984).

In order to characterise rock mass geometry for engineering purposes, discontinuity surveys include measurement of discontinuity spacing and trace length is necessary. It is certainly important to estimate these parameters with the necessary accuracy and precision by obtaining a representative and sufficient sample of spacings and trace length from the rock mass domain in question (Priest and Hudson, 1981).

For the consideration of individual rock mass discontinuities, the suggestion methods published by the International Society for Rock Mechanics (ISRM, 1981) presents 11 descriptive parameters amongst which discontinuity spacing is probably the single most important parameter, playing a great role in describing the quality of a rock mass.

The Rock Quality Designation (RQD), originally proposed by Deere (1964), is based on the spacing between consecutive open discontinuities encountered along a straight line through the rock mass. Thus the use of individual discontinuity descriptions, together with the RQD index, modified in different ways by a number of workers, has been proposed for a variety of engineering applications (Wallis and King, 1980). The discontinuity spacing will be considered with reference to the distances between points where discontinuities intersect a straight line in the rock mass. Priest and Hudson, (1976) expressed the view that the negative exponential distribution of spacing can be estimated from the following equation:

\[ f(x) = \lambda e^{-\lambda x} \]  

Mean discontinuity spacing, \( \bar{x} \): The mean value of spacings is computed as;

\[ \bar{x} = \frac{\sum x_i}{n} \]  

Where

\( x_i \) is the ith discontinuity spacing measurement along a scanline of length \( x \) yielding \( n \) values and
Where \( f(x) \) is the frequency of a discontinuity spacing \( x \), and \( \lambda \) is the average number of discontinuities per metre.

In this research, the investigation was performed in the porphyry rock mass at the Mugga II Quarry. Four separate outcrop areas of the rock mass were investigated. At each site a three dimensional joint survey was carried out and all discontinuity parameters were consider. Discontinuity spacings also were measured on the surface outcrops at the four parts of the quarry. From the total of 21 separate scan line surveys, discontinuities were read on both vertical and horizontal outcrop faces. The individual scanline lengths ranged from 4 to 30 m, covering a total length of 405 m. From the data obtained at each part of quarry, the frequency of discontinuity spacings were plotted in histogram form, using a spacing class interval of 0.1 m. Four histograms were plotted, each representing the sum data from one of the parts (Figures 5.39 to 5.42). For each data set, the least squares best fit curve was computed for a negative exponential function of the form \( f(x) = a e^{-bx} \) for \((a > 0)\), and superimposed on the histogram. The discontinuity spacing histogram for the sum of all data from Mugga II Quarry is given in Figure 5.43, together with the superimposed best fit curve. The correlation coefficient for the best fit curve is \((r^2 = 0.90)\), which clearly illustrates the random distribution of the discontinuities at this quarry. The field data and curve parameters for all four parts of the quarry are summarised on Table 5.6.

Table 5.6 Summary of field data and negative exponential curve parameters

<table>
<thead>
<tr>
<th>Sources of data</th>
<th>Total length (L (m))</th>
<th>Number of discontinuities</th>
<th>Theatrical curve parameter (( \lambda = n/L ))</th>
<th>Best fit curve parameters</th>
<th>( R^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>128</td>
<td>270</td>
<td>2.1</td>
<td>18.52</td>
<td>2.03</td>
</tr>
<tr>
<td>Section B</td>
<td>51.5</td>
<td>282</td>
<td>5.5</td>
<td>108.55</td>
<td>6.76</td>
</tr>
<tr>
<td>Section C</td>
<td>112.5</td>
<td>363</td>
<td>3.23</td>
<td>35.23</td>
<td>3.17</td>
</tr>
<tr>
<td>Section D</td>
<td>113</td>
<td>201</td>
<td>1.78</td>
<td>16.08</td>
<td>1.63</td>
</tr>
<tr>
<td>Total data</td>
<td>405</td>
<td>1116</td>
<td>2.76</td>
<td>20.50</td>
<td>2.4</td>
</tr>
</tbody>
</table>
As described in chapter 2 the spacing of a discontinuity is the perpendicular distance measured along the scan line between adjacent parallel discontinuities on the slope faces. According to the results of statistical analysis of data in section A more than 41.11% of the spacing values of the discontinuities are less than 0.20 m and all together more than 60% of discontinuity spacings are less than 0.40 m (Figure 5.39). This means that it is a heavily jointed rock mass and may have some instability problems in this particular part.

Figure 5.40 shows the results of statistical analysis of the frequency distribution of discontinuity spacings in section B of the quarry. It is clear from this figure that more than 74.8% of discontinuity spacings are less than 0.2m and 91% of them are less than 0.4m. This result shows that the distribution of joints in this part of quarry also is high.

From the results of statistical analysis of data it was concluded that more than 55.92% of the spacing values of the discontinuities are less than 0.20 m in section C (Figure 5.41). Also more than 78% of discontinuity spacings are less than 0.40 m. The mean value of the discontinuity spacings in this part was found to be 0.31m. This means that it is a very heavily jointed rock mass. The distribution of discontinuity spacings has great effect on instability of slope faces in this particular part.

In section D it is clear from the Figure 5.42 that more than 25% of discontinuity spacings are less than 0.2 m and completely 51% of them are less than 0.4 m. The mean value of discontinuity spacing in this part of quarry was found to be 0.56 m. This result shows that in this part of the quarry distribution of joints is low and it means that this part is not a very heavily jointed rock mass. Table 5.7 show frequency distribution of discontinuity spacings in different parts of quarry.

Table 5.7  Frequency distribution of discontinuity spacings in different parts of quarry.

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of Values</th>
<th>Mean</th>
<th>Std. Div</th>
<th>&lt;0.2 metre (%)</th>
<th>Over one metre (%)</th>
<th>Maximum concentration at (%)</th>
<th>Mode (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>270</td>
<td>0.47</td>
<td>0.48</td>
<td>41.11</td>
<td>14</td>
<td>0.0 - 0.2</td>
<td>41.11</td>
</tr>
<tr>
<td>Section B</td>
<td>363</td>
<td>71.19</td>
<td>17.74</td>
<td>74.82</td>
<td>0.0</td>
<td>0.0 - 0.2</td>
<td>74.82</td>
</tr>
<tr>
<td>Section C</td>
<td>282</td>
<td>60.1</td>
<td>20.64</td>
<td>55.92</td>
<td>3.4</td>
<td>0.0 - 0.2</td>
<td>55.92</td>
</tr>
<tr>
<td>Section D</td>
<td>201</td>
<td>68.19</td>
<td>21.01</td>
<td>24.88</td>
<td>20</td>
<td>0.2 - 0.4</td>
<td>26.37</td>
</tr>
</tbody>
</table>
Figure 5.39 Frequency Distribution of Discontinuity Spacings in Section A

Figure 5.40 Probability Density Histogram of Discontinuity in Section B

Figure 5.41 Relative Frequency of Discontinuity Spacings in Section C

Figure 5.36 Frequency Distribution of Discontinuity Spacings in Section D
Figure 5.43 Probability Density Histogram of Discontinuity Spacings in Mugga II Quarry

- No = 1116
- Mean = 0.36
- Std. Div = 0.41
- $a = 20.507$
- $-b = 2.4$
- $R^2 = 0.90$

Fitted negative exponential probability density distribution, $\lambda = 2.8/m$
According to the field observations this part of quarry is composed of a very good quality porphyry rock. The distribution of discontinuity sets is very limited in this part of the quarry.

5.4.1.11 Rock Quality Designation (RQD)

Rock Quality Designation is one of the first characteristics of rock mass presented by Deere (1964). For engineering purposes, due to its relative simplicity, the RQD has been extensively used for the classification of rock masses.

The estimation of RQD is based on scanline reading in the field, which requires an effective sampling procedure to yield unbiased and consistent estimates (Sen and Kazi, 1984). Deere, (1964) proposed the following equation

\[ RQD = 100 \sum_{i=1}^{n} \frac{X_i}{L} \]  

(5.11)

Where \( X_i \) denotes the length of the ith sample with length \( \geq 0.1 \) m and \( n \) denotes the number of samples with length \( \geq 0.1 \) m.

\( L \) is the total length of scanline or borehole. For the description of individual rock mass discontinuities, the Suggested Methods published by the International Society for Rock Mechanics (ISRM, 1981). In studying the distribution of discontinuity spacing along a straight line through a rock mass, Priest and Hudson (1976) have suggested that, in the absence of a large predominance of evenly spaced discontinuities, any arrangement of randomly positioned discontinuities will lead to a negative exponential form of the curve relating spacing values to frequency of occurrence. They have defined this curve as

\[ f(x) = \lambda \cdot e^{-\lambda x} \]  

(5.12)

where \( \lambda \) is the average number of discontinuities per metre. This is a one parameter (\( \lambda \)) distribution with the mean and standard deviation both equal to \( 1/\lambda \). The percentage length of the scanline or borehole consisting of spacing values greater than a given value, \( t \), gives the theoretical RQD and \( RQD^*_t \), as

\[ RQD^*_t = 100 e^{-\lambda t} (t\lambda + 1) \]  

(5.13)
Using the negative exponential distribution of discontinuity spacing values, Priest and Hudson (1976) have proposed a theoretical Rock Quality Designation (RQD*), which is determined solely from a knowledge of $\lambda$. Therefore, a relationship was established between the (RQD*) and the average number of discontinuities per metre:

$$RQD^* = 100e^{-0.1\lambda}(0.1\lambda + 1)$$  \hspace{1cm} (5.14)

With this equation it is possible to compare the theoretical RQD, obtained from the discontinuity frequency alone, with the actual RQD calculated from the spacing values. The advantage of the theoretical RQD* is that different qualifying threshold values can be employed. Priest and Hudson (1976) have shown that there is a good linear approximation to the above equation for $t = 0.1$ m and it can given by:

$$RQD^* = 110.4 - 3.68\lambda$$  \hspace{1cm} (5.15)

which provides a reasonable approximation in the range $6 < \lambda < 16$ per m. In this regard the International Society for Rock Mechanics (ISRM, 1978) proposed the following approximation empirical relationship between RQD and the Volumetric joint count $J_v$

$$RQD = 115 - 3.3J_v \text{ for } J_v \geq 4.5$$ \hspace{1cm} (5.16)

$$RQD = 100 \text{ for } J_v < 4.5$$ \hspace{1cm} (5.17)

In fact RQD provides a simple value for the considering the quality of the rock masses. The probability of discontinuity spacing occurring between $x$ and $x = dx$ is given by $f(x)dx$, where $x$ is a spacing value, $dx$ is an increment of spacing, $f(x)$ is the probability density distribution of $x$. Priest and Hudson, (1976) stated that for a long total length of scanline, $L$, the total number of discontinuity is $\lambda L$. Thus, the number of intact lengths between $x$ and $x = dx$ is $\lambda Lf(x)dx$ and the length of these is $\lambda Lxf(x)dx$. From the definition of RQD, equation 100, and in the continuous case, the theoretical (RQD*) with an arbitrary threshold value, $t$, is given by:

$$RQD^*_t = 100\int_t^\infty \lambda Lxf(x)dx / L$$ \hspace{1cm} (5.18)

and with a negative exponential distribution of spacing values, equation (5.12), the above equation will be:

$$RQD^*_t = 100\lambda^2 \int_t^\infty xe^{-\lambda x} dx$$ \hspace{1cm} (5.19)
Assessment of Slope Stability in Surface Mining of a Porphyry Rock Mass

For a long scanline, terms containing $e^{-\lambda L}$ can be ignored and:

$$RQD^* = 100e^{-\lambda (\lambda t + 1)} \quad (5.20)$$

For the conventional RQD, with a threshold value of $t = 0.1$ m, the theoretical value is:

$$RQD^* = 100e^{-0.1\lambda (0.1\lambda + 1)} \quad (5.21)$$

Different values of RQD, calculated from individual scanline and the corresponding values of $RQD^*$, calculated from theory by equation 5.15 are given in Tables 5.8 and 5.9. As is clear from Table 5.8 that for this particular porphyry rock mass, $RQD^*$ can be calculated to within 9% of the measured RQD using only the average number of discontinuity per metre ($\lambda$).

Table 5.8 Comparison between measured and theoretical RQD in a porphyry rock mass

<table>
<thead>
<tr>
<th>Source of data</th>
<th>Length (L (m))</th>
<th>Average discontinuity frequency ($\lambda$)</th>
<th>Measured RQD %</th>
<th>Theoretical RQD* %</th>
<th>Differences in RQD values (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>128</td>
<td>2.1</td>
<td>96.7</td>
<td>98.21</td>
<td>+1.51</td>
</tr>
<tr>
<td>Section B</td>
<td>51.5</td>
<td>5.5</td>
<td>80.56</td>
<td>89.6</td>
<td>+9.04</td>
</tr>
<tr>
<td>Section</td>
<td>112.5</td>
<td>3.23</td>
<td>92.14</td>
<td>96</td>
<td>+3.86</td>
</tr>
<tr>
<td>Section D</td>
<td>113</td>
<td>1.78</td>
<td>97.62</td>
<td>98.71</td>
<td>+1.09</td>
</tr>
<tr>
<td>Total</td>
<td>405</td>
<td>2.76</td>
<td>94.36</td>
<td>96.92</td>
<td>+2.56</td>
</tr>
</tbody>
</table>

The terms of rock quality percentage (RQP) and rock quality risk (RQR) were proposed by the Sen (1990). He stated that the necessity for RQP and RQR arises from two fundamental question in terms of the engineers wishing to have quantitative answers which will help them to understand better the rock mass behaviour that they dealing with. The first question is "In what percentages do each rock quality description (as very good, good, poor, and very poor) occur within the same rock mass?". In this regard, field experience indicated that fractures rocks are non-homogeneous and accordingly more than one type of rock quality may exist within the same rock mass. An answer to this question may be furnished by the RQP which is defined quantitatively as the relative frequency distribution of the classical RQD. In other words, theoretically, RQP is the
probability of a certain rock description existing in a given rock mass. The second question is "what is the risk associated with the actual RQD being less than measured RQD value which is adopted in the design criterion?". Although the value of RQD provides a certain indication, it does not provide by any means an assessment of whether this description is going to be met fully by the rock behaviour or not. In fact, RQR is defined as the probability of RQD being less than a given design value. Since, RQP and RQR are defined as probability quantities they may assume any value between 0 to 1 inclusively.

Sen (1990) used a Monte-Carlo simulation technique with the help of a computer to show the relationship between RQD, RQP and RQR. He proposed a quality description chart which enables it to consider the rock quality percentage (RQP) and also (RQR) of a rock mass (Figure 5.44). Different values of the rock quality indices calculated for the porphyry rock mass are given in Table 5.9.

Table 5.9 Different Values of RQD, RQP, and RQR for a porphyry rock mass.

<table>
<thead>
<tr>
<th>Source of data</th>
<th>Average discontinuity frequency ($\lambda$)</th>
<th>RQD</th>
<th>RQP</th>
<th>RQR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>2.1</td>
<td>96.7</td>
<td>Very good</td>
<td>0.98</td>
</tr>
<tr>
<td>Section B</td>
<td>5.5</td>
<td>80.56</td>
<td>Good - Fair</td>
<td>0.90 - 0.10%</td>
</tr>
<tr>
<td>section C</td>
<td>3.23</td>
<td>92.14</td>
<td>Very good - Good</td>
<td>0.05 - 0.95%</td>
</tr>
<tr>
<td>Section D</td>
<td>1.78</td>
<td>97.62</td>
<td>Very good</td>
<td>0.73%</td>
</tr>
<tr>
<td>Total</td>
<td>2.76</td>
<td>94.36</td>
<td>Very good - good</td>
<td>0.15 - 0.85%</td>
</tr>
</tbody>
</table>
Figure 5.44  Quality description chart (the number on the curves is $\lambda$ -- the mean number of fractures per metre). After Sen, 1990
5.4.1.12 Discontinuity trace length

The measurable length of the linear trace produced by the intersection of a planar discontinuity with a planar rock face. The end of a trace will occur either at another discontinuity or within the rock material ISRM, (1978). The end of a trace may not, however, be visible at a negative face due to excavation, erosion or the presence of vegetation covering Priest and Hudson, (1981).

The mean discontinuity trace length $\bar{l}$ can be calculated from the following equation:

$$\bar{l} = \frac{1}{n} \sum_{i=1}^{n} \frac{l_i}{n}$$  \hspace{1cm} (5.22)

where $l_i$ is the discontinuity trace length sampled in some specified way at a given rock face that yields a total of $n$ such trace lengths. Priest and Hudson (1981), defined mean trace termination frequency ($\mu$) as the reciprocal of mean discontinuity trace length and is therefore analogous to the mean discontinuity frequency. For a large sample, $\frac{1}{\bar{l}} \equiv \mu$

The probability density distribution of trace lengths over the entire rock face is denoted by $f(l)$ and the cumulative probability distribution by $F(l)$. At this stage no assumptions are made concerning the nature of $F(l)$. Priest and Hudson, (1981) suggested that if the scanline is located randomly with respect to a set of parallel discontinuity traces then the probability of the scanline intersecting a given trace is directly proportional to the length of that trace. Therefore, the probability $p(l)$, that the scanline intersects a trace with a length in the range $l$ to $l + dl$ is given by:

$$p(l) = k\, f(l)\, dl$$  \hspace{1cm} (5.23)

where $k$ is a constant.

The probability density distribution, $g(l)$ of trace lengths intersected by the scanline is therefore given by:

$$g(l) = k\, f(l) \quad \text{for} \ (l>0)$$  \hspace{1cm} (5.24)

where $g(l)$ is the probability density distribution, then
Figure 5.45 Frequency Distribution of Discontinuity Lengths in Section A

Figure 5.46. Probability Density Histogram of Discontinuity Lengths in Section B

Figure 5.47. Relative Frequency of Discontinuity Lengths in Section C

Figure 5.48. Probability Density Histogram of Discontinuity Lengths in Section D
\[
\int_0^\infty g(l)dl = 1 \quad (5.25)
\]

or

\[
k\int_0^\infty lf(l)dl = 1 \quad (5.26)
\]

but

\[
\int_0^\infty lf(l)dl = \frac{1}{\mu} \quad (5.27)
\]

therefore

\[
k = \mu \quad (5.28)
\]

and

\[
g(l) = \mu lf(l) \quad (5.29)
\]

The above equation gives the density of the biased distribution of trace lengths sampled by the scanline for any actual distribution of trace lengths. Expressions for \(g(l)\) when \(f(l)\) is a negative exponential, uniform or normal form are given in Table 5.10.

Table 5.10 Trace length distribution (After Priest and Hudson, 1981)

<table>
<thead>
<tr>
<th></th>
<th>probability density distribution</th>
<th>Population mean</th>
<th>Estimated mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trace length</td>
<td>(f(l))</td>
<td>(1/\mu)</td>
<td>(\bar{l})</td>
</tr>
<tr>
<td>Intersected trace length</td>
<td>(g(l))</td>
<td>(1/\mu_s)</td>
<td>(\bar{l}_s)</td>
</tr>
<tr>
<td>Semi-trace length</td>
<td>(h(l))</td>
<td>(1/\mu_b)</td>
<td>(\bar{l}_b)</td>
</tr>
<tr>
<td>Censored semi-trace length</td>
<td>(i(l))</td>
<td>(1/\mu_i)</td>
<td>(\bar{l}_i)</td>
</tr>
</tbody>
</table>

A frequency distribution of the lengths of discontinuities in section A of the quarry is given in Figure 5.45. Results obtained from the statistical analysis show that more than 70% of the discontinuities have a length of less than 0.5 metre and the number of
discontinuities with length of less than one metre represents 86%. This means that this parameter with reasonable length can be the cause of some sliding or instability of slope faces in this part of quarry. The mean value of the frequency distribution of discontinuity lengths in section A of the quarry was found to be 1.6.

In section B of the quarry, the results of statistical analysis of data shows that the maximum concentration of the discontinuity lengths (26.6%) belongs to discontinuities with the length between 0.5 to 1.00 metre. Table 5.11 shows the frequency distribution of discontinuity lengths with the mean value of 2.58 m in section B of Mugga II Quarry. It is clear that this factor with a considerable mean value can lead to the instability or large scale sliding on the high wall of porphyry hard rock in this part of quarry. A graphical presentation of discontinuity lengths is given in Figure 5.46.

From the results of statistical analysis of discontinuities data it was found that in this part of the quarry (part C) more than 35.6% of discontinuity lengths are more than one metre and around 19% of them have lengths of more than 5 metres. The mean value of the discontinuity lengths in section C was found to be 3.08m. This means that the rock mass is mainly governed by the joint systems with high value of length which they can influenced on the instability or sliding of slope faces in this part of quarry. A graphical presentation of discontinuity lengths is given in Figure 5.47.

From the results of statistical analysis of data in section D of the quarry it was found that the maximum concentration of the discontinuity lengths (21.39%) belongs to the discontinuities with the lengths between 1.0 - 1.5 metre. Figure 5.48 shows the frequency distribution of discontinuity lengths in section D of the quarry with the mean of 1.94. It is clear from Table 5.11 maximum concentration of discontinuity lengths are between 1 - to 1.5 metre and it can be concluded that in this part of the quarry discontinuity lengths do not have very high values in. As mentioned earlier the previous section this part of the quarry has very good quality porphyry rock mass and the only problem in this part of the quarry is the presence of a few faults in this part which can affect the instability of slope faces.
Table 5.11 Results of frequency distribution of discontinuity lengths in Sections A to D

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of Values</th>
<th>Length (L (m))</th>
<th>Mean (m)</th>
<th>Std. Div</th>
<th>&lt;0.5 m (%)</th>
<th>Over 3 m (%)</th>
<th>Maximum concentration at</th>
<th>Mode ( % )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>270</td>
<td>128</td>
<td>1.67</td>
<td>1.57</td>
<td>70</td>
<td>0.37</td>
<td>0.0 - 0.5</td>
<td>70</td>
</tr>
<tr>
<td>Section B</td>
<td>363</td>
<td>51.5</td>
<td>3.08</td>
<td>3.20</td>
<td>25.17</td>
<td>21.5</td>
<td>0.5 - 1.0</td>
<td>26.59</td>
</tr>
<tr>
<td>Section C</td>
<td>282</td>
<td>112.5</td>
<td>2.58</td>
<td>3.25</td>
<td>18.51</td>
<td>25.80</td>
<td>1.5 - 2.0</td>
<td>18.51</td>
</tr>
<tr>
<td>Section D</td>
<td>201</td>
<td>113</td>
<td>1.94</td>
<td>1.33</td>
<td>9.54</td>
<td>29</td>
<td>1.0 - 1.5</td>
<td>21.39</td>
</tr>
<tr>
<td>Total</td>
<td>1116</td>
<td>405</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

As is clear from the Table 5.11 from the Mugga II Quarry the mean Value of the discontinuity length was found to be 1.67, 3.08, 2.58 and 1.94 m respectively.

5.5 ANALYTICAL STATISTICS

5.5.1 Data Acquisition and Factor Analysis

Factor analysis is a statistical method used to identify a relatively small number of factors that can be used to represent relationships among sets of many variables. The basic assumption of factor analysis is that underlying dimensions, or factors, can be used to explain complex phenomenon (Wanger, 1992). One goal in factor analysis is to present relationship between sets of variables parsimoniously. Factor analysis usually proceeds in four steps (Norvsis, 1985). In the first step, the correlation matrix for all variables is computed.

Variables that do not appear to be related to other variables can be identified from the matrix and associated statistics. In the second step, factor extraction; the number of factors necessary to present the data and the method of calculation them must be determined. The third step, rotation, focuses on transforming the factors to make them more interpretable. In the fourth step, scores for each factor can be computed for each case. The assessment of inherent factors affecting to rock slope instability, it is necessary to classify the contributed parameters into two groups e. g independent and depended variables. In the case of rock slope stability, the stability of rock slope is function of
discontinuity parameters such as discontinuity orientations, dip angle, infilling materials, aperture, curvature, roughness, length, and discontinuity spacing which are independent variables. The effects of the above independent variables on the slope stability were examined and some 2756 data were analysed by using an advanced statistical program. When the discontinuity parameters were subjected to factor analysis, four factors yielded with eigenvalue more than one accounted for 64.6 of variance. The first factor explained 22% of the variance, the second 17.7%, the third 12.7%, and the fourth 12.5%. The results of the statistical analysis are given in Table 5.12.

From the Table 5.12 it is concluded that variables length and persistence are highly related to Factor 1 which accounts 22% of the variance. These two factors have significant contribution in the slope stability of hard rocks. After the length and persistence variables, curvature, dip angle and spacing have significant contribution in the slope stability of hard rocks.

Table 5.12  Factor loading of the stability evaluation from the contributory factors

<table>
<thead>
<tr>
<th>Independent variables</th>
<th>factor 1</th>
<th>Factor 2</th>
<th>Factor 3</th>
<th>Factor 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dip angle</td>
<td>0.13</td>
<td>-0.07</td>
<td>0.81</td>
<td>0.14</td>
</tr>
<tr>
<td>Aperture</td>
<td>0.12</td>
<td>0.58</td>
<td>0.15</td>
<td>-0.33</td>
</tr>
<tr>
<td>Curvature</td>
<td>-0.04</td>
<td>0.59</td>
<td>-0.01</td>
<td>-0.02</td>
</tr>
<tr>
<td>Roughness</td>
<td>-0.04</td>
<td>-0.21</td>
<td>-0.03</td>
<td>-0.03</td>
</tr>
<tr>
<td>Persistence</td>
<td>0.54</td>
<td>0.10</td>
<td>-0.05</td>
<td>0.10</td>
</tr>
<tr>
<td>Infill- materials</td>
<td>-0.01</td>
<td>-0.08</td>
<td>-0.55</td>
<td>0.17</td>
</tr>
<tr>
<td>Spacing</td>
<td>0.20</td>
<td>-0.08</td>
<td>0.05</td>
<td>0.91</td>
</tr>
<tr>
<td>Length</td>
<td>0.55</td>
<td>-0.03</td>
<td>-0.06</td>
<td>0.08</td>
</tr>
<tr>
<td>variance</td>
<td>0.22</td>
<td>0.18</td>
<td>0.13</td>
<td>0.13</td>
</tr>
<tr>
<td>Eigenvalue</td>
<td>1.75</td>
<td>1.42</td>
<td>1.01</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Consequently, from the statistical analysis of data it was found that most important discontinuity parameter affecting slope stability of hard rocks is the length of discontinuity which has great effects on the instability of hard rocks.

5.6 DIFFERENT MODES OF FAILURES

The simplest possible model of rock slope failure is the model of a single block sliding down the plane but, in most practical cases, a very complicating failure process occurs. In this case, the method of calculating the factor of safety is not only involved with the simple gravitational sliding. Most important modes of failure are described in the following paragraphs.

5.6.1 Plane Mode Failure

In the case of simple plane shear instability mode which arises when a discontinuity strikes parallel or nearly parallel to the slope face at a shallower dip, is shown in Figure 5.49. All the equations defining the stability of a block on an inclined plane have been presented for the condition of limiting equilibrium, i.e. the condition at which the forces tending to induce sliding are exactly balanced by those resisting sliding and the factor of safety \( F = 1 \). Therefore, when the slope is stable, the resisting forces are greater than the disturbing forces and the value of the factor of safety will be greater than 1.

![Illustration of slope geometry](image)

In this situation the factor of safety against sliding can be calculated by following equation
The normal stress $\sigma$ which acts across the potential sliding surface is given by

$$\sigma = \frac{W \cos \psi}{A}$$  \hspace{1cm} (5.31)

where; $A$ is the base area of the block. On the other hand the relationship between shear and normal stress is represented by Coulomb’s equation and can be expressed as:

$$\tau = C + \frac{W \cos \psi}{A} \tan \phi$$  \hspace{1cm} (5.33)

or

$$R = C.A + W \cos \psi \tan \phi$$  \hspace{1cm} (5.34)

where

$R = \tau.A$ is the shear surface which resists sliding down the plane. Increasing the stability of slopes by increasing the bolt force ($T$) the value of the factor of safety will be increased (Figure 5.50) and factor of safety can be expressed by the following equation;

$$Fs = R + T \cos(\psi + \beta) / W \sin \psi$$  \hspace{1cm} (5.35)

where $\beta$ is the angle between the rock bolt and sliding face. In this case the factor of safety can be expressed by

$$F = \tan \phi / \tan \beta$$  \hspace{1cm} (5.36)

This result is obtained by setting $\frac{dT}{d\beta} = 0$ and also $\frac{dF}{d\beta} = 0$.
5.6.2 Wedge Failure

The wedge failure will occur when the line of the intersection of discontinuities defining the wedge should be daylight and two discontinuities strike obliquely across the slope face. The basic mechanics of wedge failure involves the sliding of a wedge along the line of intersection of two planar discontinuities. Sometimes sliding may occur on one plane or both planes of a discontinuity depending upon their orientation.

The factor of safety of the wedge defined in Figure 5.51. Assuming that sliding is resisted by friction only and that friction angle $\phi$ is the same for the both planes is given by

$$ F = \frac{(R_A + R_B) \tan \phi}{W \sin \psi_i} $$

(5.37)

where

$R_A$ and $R_B$ are the normal reactions provided by planes A and B.

$N_a = \frac{W \cos \alpha \cos \psi_a}{\sin(\psi_a + \psi_b)} - U_a + T_a$

$N_b = \frac{W \cos \alpha \cos \psi_b}{\sin(\psi_a + \psi_b)} - U_b + T_b$

Figure 5.51 Analysing of forces affecting wedge failure

On the other hand the factor of safety for the wedge failure can be calculated by

$$ F = A \tan \phi_A + B \tan \phi_B $$

(5.38)

The above equation is valid when the cohesive strength of the plane A and B is zero and the slope is fully drained.
5.6.3 Circular Failure

This type of failure occurs, when the materials of rock is very crushed or heavily jointed. It should be noted that circular failure is more commonly failure in soils. Defining the factor of safety of the slope as

\[
F = \frac{\text{Shear strength available to resist sliding}}{\text{Shear stress mobilised along failure surface}}
\]  
(5.39)

and

\[
F = \frac{C + \sigma_n \tan \phi}{\tau_{mb}}
\]  
(5.40)

where

\( \tau_{mb} \) is the shear stress mobilised along the failure surface.

5.6.4 Toppling Failure

For the purpose of estimating the support load requirements for the toppling mode, a very simple and restricted mode was presented by Seegmiller (1981), quoted by El-Mherig (1986) as shown in Figure 5.52.

![Figure 5.52 Forces affecting the toppling failure](image)

Figure 5. 52 Forces affecting on the toppling failure
It should be noted that Hoek and Bray (1981) have made a great contribution in mathematically describing the evaluation of toppling mode failure. It is clear that toppling occurs when the center of gravity of a unit of rock overhang a pivot point within the unit as shown in Figure 5.51.

The required anchoring force at toppling limiting equilibrium of the block is given by

\[ T = 3W (\sin \alpha - \cos \alpha) + h_w \gamma_w (h_w^2 + 2b^2)/6h_a \]  

(5.41)

where

- \( W \): block weight
- \( h \): block height
- \( \alpha \): block inclination
- \( b \): block width
- \( \gamma \): block unit weight
- \( h_w \): water height

Hoek and Bray (1981) have proposed a model for limit equilibrium analysis of toppling on a stepped base. According to this limiting equilibrium the force \( P_{n-1} \) which is just sufficient to prevent toppling of the \( n^{th} \) block can be calculated by the following equation;

\[
P_{n-1} = \frac{P_n (M_n - \Delta x \tan \phi) + (W_n / 2)(Y_n \sin \alpha - \Delta x \cos \alpha)}{L_n}
\]

(5.42)

where

- \( L_n \): the point of forces acting against the toppling of nth block
- \( M_n \): the point of forces acting on the toppling of \( n^{th} \) block
- \( Y_n \): the height of the block
- \( P_n \): the force acting on the toppling of the \( n^{th} \) block
- \( P_{n-1} \): the force against the forces acting on the toppling of the \( n^{th} \) block
- \( \alpha \): the angle between the surface of toppling with the horizon
- \( \Delta x \): width of the \( n^{th} \) block
5.7 THE EFFECT OF DISCONTINUITIES IN SLOPE STABILITY

Hoek and Bray (1981) considered the effect of slope height on the slope angle in the slope stability of rocks. The results of their studies have shown that while many slopes are stable at steep angle and at height of several hundreds of feet, many flat slopes fail at height of only tens of feet. This difference is due to the fact that the stability of rock slopes varies with inclination of discontinuity surfaces, such as faults, joints and bedding planes within the rock masses. Thus when discontinuities are vertical or horizontal, simple sliding cannot take place and the slope failure will involve fracture of intact blocks of rock as well as movement along some of the discontinuities. On the other hand, when the rock mass contains discontinuity surfaces dipping towards the slope face at angles of between 30 degrees and 70 degrees, simple sliding can occur and the stability of these slopes is significantly lower than those in which only horizontal and vertical discontinuities are present.

Any discontinuities present will strongly affect the mechanical behaviour of the rock mass, in terms of its strength, deformability, fluid permeability and stability. For almost all rock engineering projects, the most important result arising from a site investigation is a comprehensive, three dimensional analysis of discontinuities present within the rock mass Wallis and King, (1980). It is clear that the presence or absence of discontinuities has a great influence upon the stability of rock slopes and these geological structures including one of the most important parts of a stability assessment in very heavily jointed hard rock slopes.

5.8 DISCONTINUITY DATA COLLECTION AND ANALYSIS

In order to study the effect of discontinuity characteristics on the overall stability of slope faces in open pit mining a joint survey program using scan - line method based on the ISRM (1981) suggestions was carried out in different accessible parts of Mugga II Quarry. For this purpose a magnetic clinometer (Clar Compass) was used for the measurement of dip and dip-direction of the discontinuity planes.

Because of the discrepancies between different characteristics of discontinuities and for the purpose of discontinuity measurement, this quarry was divided into four parts, namely part A (eastern part of quarry, bench two), part B (south east part of quarry,
bench four), part C (southern part of quarry, bench five) and finally part D (eastern part of quarry, bench six).

From the quarry all together around 1116 readings were taken while some additional data necessary for the classification of the rock mass were obtained from existing slope surfaces by the scan line method. In some areas of the quarry, especially in the eastern part (part A & D) there are some fault zones. In this part of the quarry these fault zones certainly have an influence on the stability of the slopes.

5.9 GRAPHICAL PRESENTATION OF DISCONTINUITIES

As mentioned above the measurements of discontinuity parameters such as discontinuity orientation, aperture type of discontinuity, water condition, spacing, persistence, discontinuity curvature, joint compressive strength (JCS), and joint roughness coefficient (JRC) were carried out both quantitative or qualitative using the scanline method in accordance with the ISRM (1981) recommendations.

The discontinuities measured were plotted using a computer program. The output of this program produces following plotted forms:

- Pole plot net
- Contoured pole plot nets
- Illustration of all discontinuity sets (major planes) on lower hemisphere projection.

5.9.1 Stability Assessment of the Slope Face in Section A

The stability of a slope face was assessed by using stereographic projection on data derived from a three dimensional joint survey at this part of the quarry. From the geotechnical mapping altogether 270 discontinuities in the slope faces were measured. In order to consider the slope failure potential in this part of the quarry, a lower hemisphere equal area projection method was used for the presentation of planes of discontinuities. In this slope stability analysis a slope face of 75 degrees and dip direction of 301 degrees with a friction angle of 33.6 degrees was used.
Figures 5.53 and 5.54 show the pole plot and the contour plots of discontinuities in section A of the quarry. Based on the high density pole projection of 270 discontinuities measured in the slope face of bench two, five major joint sets were identified. From Figure 5.54 (b) it can be seen that the intersection lines I_{23}, I_{24}, and I_{34} are within the shadow area and these may contribute to the wedge failures possible in this part of the quarry. Only the intersection lines of I_{15}, and I_{13} do not influence the stability of the slope face of this part of the quarry. As it is shown in Figure 5.54 (b) changing the slope face direction to 75/325 or reducing the slope angle to 70/325 may help to prevent wedge failure along the direction of the intersection line I_{23}.

5.9.2 Stability Assessment of the Slope Face in Section B

In this section of the quarry a full geological mapping of discontinuities was carried out using conventional scanline survey along the slope faces measuring all discontinuities intersected by the scanline. The stability of the slope face of this part of the quarry was carried out using a lower hemisphere equal area stereographic projection method for the presentation of major planes of discontinuities (Figures 5.55 to 5.56). Samples of naturally jointed rocks were collected during the site investigation, prepared and tested. The slope face of 75 degrees and a friction angle of 33 degrees obtained from direct shear test results were used to consider the slope failure potential in this part of the quarry. Based on the results of plotting all poles four major joint sets were identified having dips and dip directions of 78/212, 81/267, 79/01, and 10/090 respectively. In this section the slope face orientation was 75/302. From Figure 5.56 (b) it is clear that the intersection lines I_{13}, I_{23}, and I_{15} are situated within the shaded area and present wedge failure potential in this part of the quarry. To prevent instabilities the orientation of the slope face should be changed to the 60/290. It should be noted that based on the knowledge of field investigation this part of the quarry is moderately weathered and it is very heavily jointed and also a very crushed rock mass. Figure 5.56 (b) presents the stability assessment of the slope face in southeast part of the Mugga II Quarry.
Figure 5.53  Discontinuity orientation data analysis of section A from Mugga II Quarry.  (a) Pole plot net  (b) Contoured pole plot net
Figure 5.54 Analysis of cluster poles of section A from Mugga II Quarry.
(a) Superimposed a polar net on the contoured pole plot net. (b) Stereographic projection of major discontinuity planes, slope face and friction cone.
Figure 5.55  Discontinuity orientation data analysis of section B from Mugga II Quarry. (a) Pole plot net, (b) Contoured pole plot net
Figure 5.56 Analysis of cluster poles of section B from Mugga II Quarry. (a) Superimposed a polar net on the contoured pole plot net. (b) Stereographic projection of major discontinuity planes, slope face and friction cone.
5.9.3 Stability Assessment of the Slope Face in Section C

As mentioned before the quarry was divided into four sections and in each section a full investigation was carried out in order to identify the effect of slope orientations with respect to the structural geology in this porphyry rock mass. It should be noted that section C is the south part of the quarry. In this section of the quarry, stability assessment of the slope face was carried out using a lower hemisphere equal area stereographic projection method in order to present the major planes of discontinuities in this section of quarry. In this stability assessment a slope face of 75 degrees and dip direction of 308 degrees with a friction angle of 36 degrees derived from the direct shear test results of naturally jointed porphyry samples were used in order to consider the slope failure potentials. Based on the high density pole projection of 363 discontinuities (Figures 5.57 to 5.58) measured from the slope face of bench five, four dominant joint sets were identified having dips and dip directions of 70/048, 83/312, 75/340, and 81/266 respectively. The orientation of slope face in this part of quarry was 75 degrees dip and 308 degrees dip direction. As it is shown in Figure 5.58 (b) instability of this slope face may arise because of the intersection of the joint set 70/048 degrees with the joint set 81/266. It seems they are the most important joint sets which nearly lead to large scale wedge failure in this section of quarry. In addition the intersection lines of 113 and 112 are within the shaded area and these may contribute to the wedge failures possible in this part of quarry. It should be noted that based on the field investigation the porphyry rock mass has a good quality and fresh rock in this part.

5.9.4 Stability Assessment of The Slope Face in Section D

Based on the direct shear test results an angle of friction of 36 degrees and slope face of 75 degrees with a dip direction of 301 degrees were used for the stability assessment of the slope face of this section of the quarry. From the high density pole projection derived from the 201 discontinuities measured on the slope face of this section (Figures 5.59 and 5.60), four major discontinuity sets were identified having dips and dip directions of 48/272, 82/219, 31/224, and 59/356 all of which affect the rock stability. In addition two faults were identified having dips and dip
directions of 76/331, and 62/075. It can be seen from the Figure 5.60 (b) that wedge failures are possible along intersection lines I_{14}, I_{24}, I_{12}, I_{46}, I_{47}, I_{27}, I_{26}, and I_{67}, within the shadow area. It means that these faults and joint sets can lead to potential wedge failures along the intersections.

The intersection lines I_{16}, I_{17}, I_{37}, I_{36}, and I_{13} do not affect the stability of the slope face in this part of quarry. Changing the slope face direction to 75/301 can influence or prevent wedge failures along the intersection lines I_{46} and I_{24}. As mentioned in the previous sections this part of the quarry has very good quality porphyry rock and based on the field observation and also results of the statistical analysis of discontinuities data, the spacing values in this part is higher than other parts of the quarry. Table 5.13 shows the orientations of the major joint sets.

Table 5.13 Orientations of Major Discontinuity Planes and Slope Faces

<table>
<thead>
<tr>
<th>Structural Region (Dip -Dip-direction)</th>
<th>Section A</th>
<th>Section B</th>
<th>Section C</th>
<th>Section D</th>
</tr>
</thead>
<tbody>
<tr>
<td>First predominant joint set</td>
<td>17/079</td>
<td>78/222</td>
<td>68/230</td>
<td>48/272</td>
</tr>
<tr>
<td>Second predominant joint set</td>
<td>85/219</td>
<td>81/267</td>
<td>82/325</td>
<td>82/219</td>
</tr>
<tr>
<td>Third predominant joint set</td>
<td>80/354</td>
<td>78/358</td>
<td>80/342</td>
<td>31/224</td>
</tr>
<tr>
<td>Fourth predominant joint set</td>
<td>52/019</td>
<td>10/360</td>
<td>79/266</td>
<td>59/356</td>
</tr>
<tr>
<td>Fifth predominant joint set</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Fault 1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>76/331</td>
</tr>
<tr>
<td>Fault 2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>62/075</td>
</tr>
<tr>
<td>Slope face</td>
<td>75/297</td>
<td>75/207</td>
<td>75/268</td>
<td>75/301</td>
</tr>
<tr>
<td>Changed slope face</td>
<td>75/325</td>
<td>60/200</td>
<td>64/308</td>
<td>75/290</td>
</tr>
</tbody>
</table>
Figure 5.57 Illustration of discontinuity orientation data of section C from Mugga II Quarry. (a) Pole plot net. (b) Contoured plot net.
Figure 5.58 Analysis of cluster poles of section C from Mugga II Quarry. (a) Superimposed a polar net on the contoured pole plot net. (b) Stereographic projection of major discontinuity planes, slope face and friction cone.
Figure 5.59 Analysis of discontinuity orientation data of section D from Mugga II Quarry. (a) Pole plot net. (b) Contoured plot net.
Figure 5.60  Analysis of cluster poles of section D from Mugga II Quarry.  
(a) Superimposed a polar net on the contoured pole plot net.  (b) Stereographic projection of major discontinuity planes, slope face and friction cone.
5.10 ENGINEERING PROPERTIES OF INTACT ROCK

The detail of methods of engineering properties considering of intact rock samples from the quarries were described in the chapter three. Most important properties of intact rocks carried out from laboratory tests on different types of samples are given in Table 5.14. The engineering properties of intact rock were used in the consideration of rock mass strength and also in the rock mass classification as input data.

Table 5.14 Engineering properties of porphyry intact rock from Mugga II Quarry

<table>
<thead>
<tr>
<th>Engineering properties of intact rock</th>
<th>Structural region A and B</th>
<th>Structural region C and D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial compressive strength (MPa)</td>
<td>130.25</td>
<td>186.35</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>7.36</td>
<td>12.38</td>
</tr>
<tr>
<td>UCS derived from point load test</td>
<td>97.26</td>
<td>195.50</td>
</tr>
<tr>
<td>(diametral)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UCS derived from point load test</td>
<td>60.29</td>
<td>209.29</td>
</tr>
<tr>
<td>(axial)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poisson's ratio (MPa)</td>
<td>0.22</td>
<td>0.25</td>
</tr>
<tr>
<td>Elastic modulus (GPa)</td>
<td>38.5</td>
<td>46.2</td>
</tr>
<tr>
<td>Friction angle</td>
<td>52.1</td>
<td>54.65</td>
</tr>
<tr>
<td>Bulk density kg/m³</td>
<td>2627.20</td>
<td>2660.93</td>
</tr>
</tbody>
</table>

5.11 EVALUATION OF ROCK MASS STRENGTH PARAMETERS

As it was described in detail in chapter four, in order to evaluate the shear strength parameters of the rock mass, some naturally jointed samples were collected from the slope faces of the quarry as representative angle of friction. The shear strength parameters can be estimated by laboratory and field tests. In this research work, because of some difficulties such as the high cost of field test the laboratory tests have been carried out on the naturally jointed samples of porphyry rocks and the shear strength parameters were obtained. Based on the direct shear test results friction angle for sections A and B was found to be 33.6 degrees and it was found to be 36 degrees for sections C and D. The amount of cohesive strength for sections A and B was found to be 2.4 Mpa and it was found to be 4.2 Mpa for
sections C and D. It should be noted that based on the field observation and from point of view of engineering geology the type of rocks in sections A and B was moderately weathered porphyry rock and in sections it was fresh and unweathered for sections C and D. Therefore the collected and tested samples for these sections were based on the variation of the lithology in different parts of the quarry.

5.12 ROCK MASS CLASSIFICATION SYSTEMS AT MUGGA II QUARRY

For the purpose of characterizing rock mass at Mugga II Quarry the geomechanics classification (RMR), Norwegian Geotechnical Institute (NGI) system (Q system), and Weakening Coefficient system (WC) were used. The assessment of the geotechnical parameters of porphyry rock including; uniaxial compressive strength of intact rock (UCS), rock quality designation (RQD), joint condition, joint spacing, ground water condition and the effect of the orientation of joints on the stability of slope faces in different parts of Mugga II Quarry were carried out according to the following descriptions and summarized in Table 5.15. The values of uniaxial compressive strength were obtained from the laboratory tests of intact rock sample representative of different parts of the quarries. The amounts of the RQD were calculated from the discontinuities survey and the means of joint spacing were obtained from the statistical analysis of discontinuities data. The ground water condition was considered in field investigations and in most part of the quarry it was found to be dry condition as it was discussed in the statistical analysis section. The effect of discontinuity orientations on the slope stability for each part of the quarries was considered.

5.12.1 Application of Rock Mass Quality System (RMR)

It is clear from the section A of the Table 2.10 that, five geotechnical parameters are grouped into five ranges of values. The porphyry rock mass in the quarry was divided into four structural regions having certain common features. The first five classification parameters for each region were determined according to the actual field survey and values were quantified according to the ranges of values. The
rating from the five parameters are then summed to yield the basic rock mass rating for individual structural region. The next step is to include the influence of orientation of discontinuities by adjusting the basic rock mass rating. In this stage the rating adjustments of joint orientation were quantified according to the section B of Table 2.10. The RMR rating values for the different sections of the quarry are given in Table 5.15

In addition to the RMR index, the Norwegian Geotechnical Institute (NGI) system (Q index) proposed by Barton (1974), and Weakening Coefficient classification systems (WC) propose by Singh (1986), were also examined for evaluating the rock mass quality

The results from geomechanics rock mass classification system (RMR) as presented in Table 5.15 showed that the quality of porphyry rock mass in different parts of the Mugga II Quarry can be classified as fair rock. It should be noted that, field observations and laboratory testing of the intact rock samples from sections A to D showed that sections A and B consists of a weathered porphyry rock and sections C and D have a good quality porphyry rock mass.
### Table 5.15 Geomechanics Classification (RMR) of Porphyry Rock Mass in Mugga II Quarry

| Rock Mass Parameters | Section A | | Section B | | Section C | | Section D |
|----------------------|-----------|----------------|-----------|----------------|-----------|----------------|
|                      | Value     | Rating         | Value     | Rating         | Value     | Rating         | Value     | Rating         |
| U. C. S. (MPa)       | 130.25    | 12             | 130.25    | 12             | 186.35    | 12             | 186.35    | 12             |
| RQD %                | 96.7      | 20             | 80.56     | 17             | 92.14     | 20             | 97.62     | 20             |
| Discontinuity Spacing (m) | 0.47 | 10             | 0.18      | 8              | 0.31      | 10             | 0.56      | 10             |
| Discontinuity Condition | Class II | 25             | ClassIII | 20             | Class II | 25             | Class II | 25             |
| Ground Water Condition | Dry      | 15             | Dry       | 15             | Dry       | 15             | Damp      | 10             |
| Orientation Rating   | Class III | -25            | ClassIII | -25            | Class III | -25            | Class III | -25            |
| Total RMR Rating     | 57        |                | 47        |                | 57        |                | 52        |                |

### Table 5.16 Rating Adjustment for Discontinuity orientation

<table>
<thead>
<tr>
<th>Discontinuity orientation</th>
<th>No modes of failure</th>
<th>Potential mode of failure</th>
<th>One mode of failure</th>
<th>Two modes of failure</th>
<th>Several modes of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rating</td>
<td>0</td>
<td>-5</td>
<td>-25</td>
<td>-50</td>
<td>-60</td>
</tr>
</tbody>
</table>
5.12.2 Application of Q System

The Norwegian Geotechnical Institute (NGI) system (Q index) was also examined in this quarry. The Q system of classification of rock mass is based on six parameters which are used in the description of rock mass quality, and these are include:

R.Q.D (rock quality designation).

J_n = number of joint sets.

J_r = rating of joint roughness.

J_a = joint alteration number (degree of alteration).

J_w = joint water reduction number.

S.R.F. or stress reduction factor.

The results of Q classification system of rock mass is given in Table 5.16.

As is clear from Table 5.16 the porphyry rock mass showed a range of very poor to poor quality in sections A to D respectively. According to field observations the quality of porphyry rock mass in sections A and B is poor, but in sections C and D it has a good quality rock.

5.12.3 Application of Weakening Coefficient Classification System (WC)

Weakening Coefficient is a rock mass classification system proposed by Singh (1986) and modified by the author was used as a rock mass classification system for the evaluation of quality of porphyry rock mass in Mugga II Quarry. This system was developed to define a reduction factor, weakening coefficient (WC) which could be applied to the intact samples values for determination of rock mass properties. This system of classification including the five geotechnical parameters and rating adjustment for orientation of discontinuities are given in Table 2.12. The results of WC classification system for different sections of Mugga II Quarry are given in Table 5.17.
<table>
<thead>
<tr>
<th>Rock Mass Parameters</th>
<th>Structural region (A)</th>
<th>Structural region (B)</th>
<th>Structural region (C)</th>
<th>Structural region (D)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value</td>
<td>Rating</td>
<td>Value</td>
<td>Rating</td>
</tr>
<tr>
<td>Rock Quality Designation (RQD) %</td>
<td>96.7</td>
<td>96.7</td>
<td>80.56</td>
<td>80.56</td>
</tr>
<tr>
<td>Joint Set Number (Jn)</td>
<td>5</td>
<td>15</td>
<td>5</td>
<td>15</td>
</tr>
<tr>
<td>Shear Strength Factor (J_r / J_s)</td>
<td>1.5/2</td>
<td>0.75</td>
<td>1/2</td>
<td>0.5</td>
</tr>
<tr>
<td>Joint Water Reduction Factor (J_w)</td>
<td>Dry</td>
<td>1</td>
<td>Dry</td>
<td>1</td>
</tr>
<tr>
<td>Rating Adjustment Factor (RAF)</td>
<td>5</td>
<td></td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Total rating</td>
<td>0.967</td>
<td></td>
<td>0.537</td>
<td></td>
</tr>
<tr>
<td>Description</td>
<td>Very poor</td>
<td></td>
<td>Very poor</td>
<td></td>
</tr>
</tbody>
</table>
Table 5.17. The Results of Weakening Coefficient Classification (WC) of Porphyry Rock In Mugga II Quarry

<table>
<thead>
<tr>
<th>Rock mass parameters</th>
<th>Section A</th>
<th>Section B</th>
<th>Section C</th>
<th>Section D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value</td>
<td>Rating</td>
<td>Value</td>
<td>Rating</td>
</tr>
<tr>
<td>RQD %</td>
<td>96.7</td>
<td>0.8</td>
<td>80.54</td>
<td>0.8</td>
</tr>
<tr>
<td>Discontinuity Spacing (m)</td>
<td>0.47</td>
<td>0.8</td>
<td>0.18</td>
<td>0.17</td>
</tr>
<tr>
<td>Joint Surface</td>
<td>Rough</td>
<td>0.9</td>
<td>Smooth</td>
<td>0.8</td>
</tr>
<tr>
<td>Joint Infilling</td>
<td>Open &lt;5mm</td>
<td>0.7</td>
<td>Open &gt;5mm</td>
<td>0.6</td>
</tr>
<tr>
<td>Discontinuity Aperture</td>
<td>&lt;2mm</td>
<td>0.7</td>
<td>6-20</td>
<td>0.5</td>
</tr>
<tr>
<td>Weakening Coefficient (WC)</td>
<td>0.29</td>
<td></td>
<td>0.13</td>
<td></td>
</tr>
<tr>
<td>Orientation Rating</td>
<td>0.29 × 0.37</td>
<td></td>
<td>0.13 × 0.37</td>
<td></td>
</tr>
<tr>
<td>Total Weakening Coefficient (WC)</td>
<td>0.11</td>
<td></td>
<td>0.048</td>
<td></td>
</tr>
<tr>
<td>Description</td>
<td>Moderate</td>
<td></td>
<td>Poor</td>
<td></td>
</tr>
</tbody>
</table>

- RQD %: Rock Quality Designation.
- Discontinuity Spacing (m): Distance between discontinuities.
- Joint Surface: Condition of joint surfaces.
- Joint Infilling: Type and degree of infilling.
- Discontinuity Aperture: Width of discontinuities.
- Weakening Coefficient (WC): A measure of the susceptibility of rock mass to weakening.
- Orientation Rating: Assessment of the orientation of discontinuities.
- Total Weakening Coefficient (WC): Summation of all weakening factors.
From Table 5.17 it is clear that, the porphyry rock mass showed a moderate to poor quality in sections A and B and poor to moderate quality in sections C and D respectively. As it was discussed in the previous section, sections A and B consist of a weathered porphyry rock and sections C and D have a fresh and good quality porphyry rock mass. As a result, it can be concluded that, results of rock mass classifications are not exactly compatible with to the results of field observations. Therefore it seems that the rating given to the different parameters in these rock mass classifications are not very consistent with the realistic behaviour of rock and field condition of rock masses.

5.13 ENGINEERING DESCRIPTION OF THE PORPHYRY ROCK MASS IN MUGGA II QUARRY

Laboratory testing and field investigations in Mugga II Quarry provided a range of valuable information which can be used as input data in rock engineering project in the porphyry rock mass. Based on joint survey program, laboratory testing and rock mass classification systems, a summary of the most important engineering properties and physical characteristics of porphyry rock from Mugga II Quarry are presented in Table 5.18.

Table 5.18 Engineering description of porphyry rock mass in Mugga II Quarry

<table>
<thead>
<tr>
<th>Property</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>Compact blue-gray quartz-feldesparr porphyry</td>
</tr>
<tr>
<td>Rock mass</td>
<td>Typically massive, partly bedded in southeast part, weathered in Sections A and B (East and South)</td>
</tr>
<tr>
<td>Rock Quality Designation (RQD)</td>
<td>96.7 and 80.56 in sections A and B and 92.14 and 97.62 in sections C and D respectively</td>
</tr>
<tr>
<td>Joint set numbers and orientations</td>
<td>Four major discontinuity sets in sections A to D</td>
</tr>
<tr>
<td>Joint roughness</td>
<td>Rough to smooth in section A, C, and D. Smooth to rough in section B</td>
</tr>
<tr>
<td>Water condition</td>
<td>Mainly dry in section A, B, and C. Damp in section D</td>
</tr>
<tr>
<td>Rock mass rating and quality</td>
<td>47 - 57 in sections A to D (fair quality)</td>
</tr>
<tr>
<td>Cohesion and friction angle of the rock mass (shear strength parameters)</td>
<td>200 - 300 kPa and 25° - 35° in sections A to D</td>
</tr>
</tbody>
</table>
5.14 CONCLUSIONS

The research carried out showed that geotechnical investigations together with structural analysis provides a practical method for the assessment of stability of high wall slope faces in hard rock quarries. Based on the stability analysis of the slope faces, four dominant joint sets were identified in sections A to D of the quarry. It has also been shown that simple remedial measures including changes in the direction of the quarry slope face may improve stability.

Stability assessment of the porphyry rock mass in different parts of the quarry showed that the orientation of discontinuities, geological structures and orientation of slope faces or orientations of excavation are the most important factors affecting the stability of hard rock slopes. The results of stability analysis showed that there is a potential of wedge failure in sections A to D of the quarry.

Statistical techniques were used in order to study the importance of the discontinuities on the overall stability of slope faces in open pit mining operations and results showed that between discontinuities contributing on the stability of hard rock slopes, the length of the discontinuity is the most important factor acting on the stability of hard rock slopes in open pit mines.

Results of direct shear tests shows that the shear strength parameters (friction angle and cohesive strength of the discontinuities) have a significant influence on the stability of the slope faces. Result of different rock mass classification showed that the present rock mass is very poor to moderate in sections A and B. It also showed a poor to moderate quality in sections C and D. The values of the rock mass cohesion and friction angle for the porphyry rock mass were found to be 200 - 300 kPa and 25° - 30° in sections A to D respectively. Consequently the porphyry rock mass is a heavily jointed rock mass with the poor to moderate quality.
Chapter 6
Geological Investigation and Apprasial of Slope Stability of a Limestone Quarry
CHAPTER SIX

GEOLOGICAL INVESTIGATION AND APPRAISAL OF
SLOPE STABILITY OF A LIMESTONE QUARRY

6.1 INTRODUCTION

This chapter presents a case history regarding assessment of instability of high wall slopes in a very heavily jointed rock mass at Marulan Quarry. For this purpose, laboratory and field investigations were carried out in order to collect require input data for the aim of slope stability analysis. The methodology of field investigation for the assessment of geotechnical parameters affecting stability of slope faces was based on a three dimensional joint survey of the discontinuities from the slope faces of the Marulan Quarry. The main emphasis was given to the collection, compilation and evaluation of engineering geological, discontinuity characteristics and rock mechanics data of a highly fractured and semi weathered limestone rock mass at Marulan Quarry.

The geotechnical assessment of the instability of high wall slopes in a very heavily jointed rock mass included the statistical analysis of discontinuity data obtained from geological mapping of slope faces was carried out in order to study the significance of the discontinuity parameters and their relationships to mechanical properties of jointed rock mass. The application of the stereographic projection technique on the stability analysis of the hard rocks is also presented.
6.2 GENERAL GEOLOGY AND SITE INVESTIGATIONS

Bungonia lies approximately 150 km Southwest part of Sydney near the western margin of the southern Sydney Basin. Figure 6.1 shows study area in the south-west of Sydney. Carr et al, (1980) reported that rocks in this area range in age from Ordovician to Devonian with a few erosional remnants of younger sedimentary and volcanic rocks. The dating of rocks from the Bungonia District was carried out by Carr et al, (1980), using the K-Ar dating method. Results showed that the lower part of Bungonia Limestone belongs to the late Silurian period. This geological information (Bungonia region) includes a sequence of the Paleozoic age (Ordovician to Devonian) containing sedimentary and volcanic rocks.

The Marulan Batholith and erosional remnants of the younger sedimentary and volcanic rocks overlie part of the Bungonia area. The geological history of the Bungonia area is contained in Woolnough's research, (1909) (quoted by Carr et al, 1980.). The geology of the Marulan-Bungonia district has been studied by many researchers such as; Naylor (1936, 1939, 1950, 1935), and Carr et al (1980,1981).

Figure 6.1 Illustration of study area and regional setting of the Bungonia are in the eastern Lachlan Fold Belt (After, Carr et al, 1983)
6.3 GEOLOGY OF THE MARULAN QUARRY

From a geological point of view, Bungonia-Marulan is a complex area and has been the subject of many studies over the years (Sevenson (1950), Flinter (1950), Gould (1966), Robinson (1972), Counsell (1973), quoted by Longworth & McKenzic Ltd (1976)). The Marulan quarry is located 10 Km east of the Hume Highway, in the south-east of Sydney, in the Bungonia region. A simplified geological map of the Marulan-Windellama area is presented in Figure 6.2.

"The geological province within which Marulan is located, was formed as part of the Capertee High structural zone, within the massive Lachlan Geosyncline. Widespread marine environments prevailed, but Paleozoic sedimentation was variable and complex. Thick sedimentary sequences are laid down, volcanic associations were introduced and tectonic activity was pronounced, throughout this region of Southeast New South Wales", Longworth & McKenzic Ltd, (1976).

The Paleozoic outcroppings around the South Marulan and Bungonia Canyon, include a number of sedimentary rocks with a north/south strike, generally parallel, which comprise the Bungonia Limestone Group. The Bungonia Limestone Group is bounded by the Tallong beds in the east (which are the oldest known rocks in the Bungonia region). The Bungonia Limestone Group is truncated by Glenrock Granodiorite from the north which is substantially weathered due to surface exposure.

Generally the Bungonia Limestone Group consists of a sequence of slates, shales, phyllites, sandstones, siltstones, and quartzites which are tightly folded as it is shown in Figure 6.3. In most parts of the area these rocks are thinly bedded and fossiliferous, but preservation is poor. The Bungonia Limestone Group passes into the Tangerang Volcanic region to the west. The (Tangerang Volcanic) extrusive series consists of lavas, toscanites, tuffs, and tuffaceous sandstones, with a similar strike and dip to the lower strata (Longworth & McKenzic Ltd, 1976).
Figure 6.2 Illustration of the geology of the Marulan-Windellama area showing the main features of the Paleozoic succession (After Carr et al., 1981).
Figure 6.3 Geological map of the Marulan South-Bungonia Gorge-Carne area (After Carr et al, 1983).
The Bungonia Limestone Group is subdivided into five units namely lower limestone, lower shale, middle limestone, upper shale and upper limestone respectively. A marked angular discordance between the two formations in Bungonia Creek has been described as an angular unconformity (Woolnough, 1909), and Robinson, 1940). In addition it has been confirmed that the boundary is an angular unconformity in the same areas and a fault in other areas (Carr et al, 1981).

6.4 STRATIGRAPHY OF BUNGONIA LIMESTONE

The lower limestone unit forms the basal part of the Bungonia Limestone formation, except on the western part of the syncline, northeast of Carne, where it is highlighted by a thin fossiliferous sandstone layer. Because of the intrusion of the Glenrock Granodiorite, which is a pluton of the Marulan Batholith, the Marulan south region has been extensively recrystallized (Carr et al, 1981).

Basal lower limestone has been characterized by the presence of a large number of pentamerid brachiopods. Bryozoans, crinoids and corals are a regular feature, brachiopods occur in small numbers and stromatolites occur in greater numbers at the base and are less common towards the top of the units, Carr et al (1981). The next unit comprises lower shale with thicknesses varying from 300 m in the Bungonia and Bungonia Gorge regions to 120 m around Carne. A simplified stratigraphic column of the Bungonia region is shown in Figure 6.4, which illustrates the maximum thickness (m) of the late Silurian and early Devonian units.

According to the Carr et al (1981) report, a thin, less than 20 cm interbed of graded sandstone containing angular clasts (up to 5 cm) of shale, limestone and fossil fragments, occurring within the siliceous shale near the top of the lower shale unit.

The middle limestone unit contains variably thicknesses from 250 m around Carne to 110 m northwards and 60 m southwards. Carr et al (1981) have reported that the middle limestone has a well-developed flaggy bedding with biomicrite and biosparite interbedded with micrite and fossiliferous micrite. Small lenses of biosparudite are also present.
Figure 6.4 Stratigraphic column for the Bungonia area showing maximum thickness (m) of the Late Silurian and Early Devonian units (After car et al, 1983).
The next unit is upper shale, which occurs in the area between Carne and Marulan south. It includes shale ranging from 50 m to 110 m thick. Finally, upper limestone forms the top section of the Bungonia Limestone Group which and occurs in the northern part of the Carne area. This unit is of variable thickness, from 100 m to 5 m towards the southern sector.

6.5 MAIN LIMESTONE BODY

The main limestone body is typically thin rising to thickly bedded, comprising light to dark gray limestone which is recrystallized and medium grained. The degree of weathering is moderate, but is locally variable in different sections. Traces of karstifications and cavitation within the limestone body is often present, as has been reported by other researchers (in thesis work). Cavities occur throughout the limestone in different sizes, some of them are of considerable size and are filled with different materials including clay, gravels, breccia, limestone, limonite, etc.

According to Longworth & McKenzic Ltd, (1976), the main limestone body is an elongated block with an outcropping width of around 260 m. This body dips almost vertically in the northern end of the quarry area, but flattens to a dip of around 70° south west. Figure 6.5. shows a general view of the Marulan Limestone quarry. As shown in this photograph, Marulan quarry is a large quarry which has been worked for more than 90 years. In the northern part of the quarry, seven benches have been excavated over a period of many years and includes very good quality Calcium Carbonates. The main limestone body in Marulan is of predominately high grade Calcium Carbonates, containing more than 95 % Ca CO₃. However, it also contain variably distributed contaminants. it should be noted that the main limestone body within the Marulan quarry is equivalent to Eastern, Lower, White, or Lookdown Limestone and it is the lowest unit within the Bungonia Limestone Group.

A normal high grade limestone contains up to 3% of MgCO₃, while a number of lithological variants contain higher percentages of Magnesium Carbonate, and many exceed the acceptable limit for blending with normal grade limestone for use in cement manufacture.
Figure 6.5 General view of the large limestone quarry (Marulan Quarry).
Note that the most widespread rock types, which are potentially magnesium, are limestone breccias and conglomerates, which commonly contain up to 6% and occasionally in excess of 8% MgCO3. These types of rock are unevenly distributed throughout the quarry area, and are usually discontinuous along the strike, becoming common towards the south. The production from Marulan quarry is used mainly by BHP Ltd in steel making, low shrinkage cements, and also for some chemical purposes.

A number of dolerite dykes, usually less than one metre thick, occur within the quarry, but a number of dykes more than 1 metre thick are located in the northeast part of the quarry, as shown in Figure 6.6. It should be noted that some of these dykes are highly weathered (north walls of the quarry dykes) and some are fresh and unweathered (west walls of the quarry). Generally, the trend of these dykes are approximately east-west and some north-south.

### 6.6 GEOHYDROLOGICAL CONDITION

Groundwater has been encountered in two boreholes in the central and southern parts of the quarry, at a level of 480 m in the Mt Frome Limestone and at the 360 metre level in the Eastern Limestone. Reduced permeability within the Eastern Limestone, below the 360 m level, is also indicated by small springs which emerge at about this level on the slopes above Bungonia Creek. Shallow water is also known to occur in the eastern Limestone at the extreme northern section of the quarry. Here, the basic dyke crossing the limestone forms an impermeable barrier to the southward movement of water. The water encountered in the Mt Frome Limestone is believed to be a perched water table of only very local significance. It is not considered that groundwater will be a serious problem during quarry operations.
6.7 RESULTS OF THE FIELD OBSERVATIONS AND GEOLOGICAL STUDIES

Based on the field observations and geological studies it was concluded that the limestone rock mass in Marulan Quarry is a recrystallised and medium grained, light to dark gray limestone and it is a thickly bedded, high quality, and semi weathered rock mass. Due to the presence of some major discontinuity sets and a few dolerite dykes which are mainly weathered in the north part of the quarry, it is a very heavily jointed rock mass as is presented in Figure 6.6.

6.8 METHODOLOGY OF DISCONTINUITY DATA COLLECTION AND ANALYSIS

The methods of data collection from discontinuities of slope faces in Marulan Quarry was based on suggestions of ISRM, (1981). For the stability assessment of the slope faces of the Marulan Quarry it was divided into two A and B sections. Geological and
geotechnical field investigations were carried out in order to study main type of rocks and geological features. A three dimensional joint survey program was carried out in order to collect geotechnical data from the discontinuities of the slope faces. Some 768 reading were taken from the discontinuities which intersected the scan-line. Discontinuity data collected were analysed using statistical analysis techniques and were also used as input data for the evaluation of the rock mass quality using different rock mass classifications.

6.9 STATISTICAL ANALYSIS OF DISCONTINUITY DATA FROM MARULAN QUARRY

6.9.1 Introduction

Statistical analysis methods was employed to quantitatively and qualitatively evaluate the variations of discontinuity data as well as laboratory data for the purpose of rock engineering projects. Since the statistical analysis of discontinuity data provides very high value information regarding the behaviour of rock mass discontinuities and their interactions, it is necessary to use suitable statistical methods for the analysis of discontinuity data.

6.9.2 Descriptive Statistical Analysis

As mentioned previously, the discontinuity data collected from geological mapping and site investigations from a limestone quarry were classified as numerical data in order to ascertain the frequency distribution of discontinuities in this particular rock mass. The detail of discontinuity data analysis in Marulan Quarry are presented in the following paragraphs.

6.9.2.1 Discontinuity Orientations

The results presented are based on data collected though a joint survey at the Marulan Quarry. Two different parts of the limestone quarry were examined in order to assess the potential of modes of failure in these parts of the rock mass. The results of statistical analysis of frequency distribution of discontinuity orientations from the quarry is given in
Table 6.1. Figures 6.7 and 6.8 shows the rose diagrams of sections A and B. These figures show significant clusters of preferred dip directions of discontinuities in these parts of the rock mass.

Table 6.1 Results of Descriptive Statistical Analysis of Discontinuity Orientations data from Marulan Quarry

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of values</th>
<th>Mean</th>
<th>Std. Div</th>
<th>Minimum (degree)</th>
<th>Maximum concentration at (degree)</th>
<th>Mode ( % )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>357</td>
<td>179.24</td>
<td>56.58</td>
<td>80</td>
<td>180 - 200</td>
<td>14.28</td>
</tr>
<tr>
<td>Section B</td>
<td>411</td>
<td>197.48</td>
<td>71.19</td>
<td>40</td>
<td>100 -- 120</td>
<td>21.17</td>
</tr>
</tbody>
</table>

As is clear from the Table 6.1, the orientation of discontinuities have a high range of variation in Sections A and B of the quarry.

6.9.2.2 Dip angle of discontinuities

The dip angle of discontinuities has a great effect on the stability of slope faces and also on the economical operation of any open pit mine. In this regard, the frequency distribution of dip angle of discontinuities were examined using statistical methods. The results are presented in the forms of histograms in Figures 6.9 and 6.10. The histogram in Figure 6.9 (section A) based upon 357 discontinuity data shows that 32.77 % of discontinuities have an angle of dip between 70 - 80 degrees and more than 62 % of discontinuities have an angle of dip greater than 70 degrees.

In the case of section B, more than 16.5 % of discontinuities have an angle of dip between 70 - 80 degrees and 52 % of discontinuities have an angle of dip more than 70 degrees. The results of statistical analysis of the discontinuity dip angle is given in Table 6.2. It should be noted that these high values of dip angle of discontinuities can lead to the instability of slope faces of the quarry, as was concluded in the stability assessment of the slope faces which is discussed in detail in section 6.11.
In this section, some new dip angle and orientation have proposed in order to reduce the potential of failure in these parts of the quarry.

6.9.2.3 Discontinuity aperture

The discontinuity aperture is the amount of opening between two discontinuous walls of joint plates which can be directly measured in joint survey programs. The mechanical aperture is usually generated as a result of geological shear displacement along an irregular discontinuity surface. At the Marulan Quarry the frequency distribution of discontinuity apertures were examined and the results of statistical analysis are shown as a histogram. Figure 6.11 shows the frequency distribution of discontinuity apertures in section A of the quarry. As is clear from this figure, 69.18% of discontinuity apertures have an aperture between 2 - 6 mm. In section B, 72.01 of discontinuity apertures have an aperture of between 2 - 6 mm. Figure 6.12 represents a graphical presentation of frequency distribution of discontinuity apertures in section B. It should be noted that, although this limestone rock mass is very heavily jointed, and intersected by many dolerite dykes, the class of aperture is not of a high order e.g most parts of discontinuities have an aperture of less than 6 mm.

6.9.2.4 Discontinuity infilling materials

The effect that infilling material has on slope stability was discussed in detail in the previous chapter. From the results of statistical analysis of data in section A it was found
that 48.44% of discontinuities show staining. For section B, the results show that more than 54% of discontinuities exhibit staining. It should be noted that different types of infilling materials were collected and tested, using the X-RD method in order to ascertain their composition. The results of X-RD are given in Appendix 1. The frequency distribution of infilling materials from section A and B are presented in Figures 6.13 and 6.14 respectively.

6.9.2.5 Discontinuity water condition

As was discussed in chapter two, knowledge of the pore water pressure is of utmost importance for assessing the stability of open pit mines. Some of the natural processes such as, mineralisation and aquifer recharge together with technological processes such as water, oil and gas recovery rely upon the contribution made to mass permeability by the discontinuity network. The water permeability of most rock materials lies in the range $10^{-10}$ to $10^{-15}$ ms$^{-1}$ (Louis, 1969, quoted by Priest, 1993).

The water condition in the limestone quarry was considered based on the field observations and previous reports conducted by other researchers. According to the results of statistical analysis of field data for section A of the quarry, it was found that most discontinuities (98.60%) are in dry condition. In section B of the quarry, 78.10% of discontinuities are in dry condition and 21.90% show some seepage traces. Table 6.3 represents the frequency distribution of water condition in sections A and B of Marulan Quarry.

Table 6.3 Results of statistical analysis of discontinuity water condition field data in sections A and B of Marulan Quarry.

<table>
<thead>
<tr>
<th>Categories</th>
<th>Section A</th>
<th>Section B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>Dry</td>
<td>352</td>
<td>98.60</td>
</tr>
<tr>
<td>Seepage</td>
<td>5</td>
<td>1.40</td>
</tr>
<tr>
<td>&lt;0.1 l/sec</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Total Values</td>
<td>357</td>
<td>100</td>
</tr>
</tbody>
</table>
Figure 6.7  Rose diagram showing 357 joint terraces in Section A

Figure 6.8  Rose diagram showing the orientation in Section B

Figure 6.9  Probability Density Histogram of the Plane Inclination in Section A

Figure 6.10  Frequency Distribution of Discontinuity Dip Angle in Section
Figure 6.11 Probability Density Histogram of Aperture in Section B

Figure 6.12 Frequency Distribution of Discontinuity Aperture in Section B

Figure 6.13 Probability Distribution of Discontinuity Infilling Materials in section A

Figure 6.14 Frequency Distribution of Discontinuity Infilling Materials In section B
6.9.2.6 Discontinuity Curvature

An examination of the curvatures and waviness of the discontinuities within the limestone rock mass was carried out according to ISRM, 1981 recommendations. The results of statistical analysis showed that, in section A, there is a balance between stepped and planar categories as is shown in Figure 6.15. In the case of section B, 34.31% of discontinuities have planar curvatures. Figure 6.16 presents a histogram of the frequency distribution of discontinuity curvatures in section B. The results of statistical analysis of field data are given in Table 6.4.

Table 6.4 Results of statistical analysis of discontinuity curvatures in limestone rock mass.

<table>
<thead>
<tr>
<th>Categories</th>
<th>Section A</th>
<th>Section B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>Stepped</td>
<td>163</td>
<td>45.65</td>
</tr>
<tr>
<td>Undulating</td>
<td>40</td>
<td>11.20</td>
</tr>
<tr>
<td>Planar</td>
<td>154</td>
<td>43.14</td>
</tr>
<tr>
<td>Total Values</td>
<td>357</td>
<td>100</td>
</tr>
</tbody>
</table>

As is clear from the Table 6.4 most of the discontinuities showed stepped form of the curvature. It can helped to the increasing of the discontinuities resistance against the sliding of the slope faces in these parts of the quarry.

6.9.2.7 Discontinuity roughness

A full explanation of the quantitative description of joint roughness and its effect on the shear strength behaviour and dilation of joints was expressed in chapter four of this thesis. For the assessment of small scale discontinuity roughnesses in marulan quarry, a contour gauge was used. From the results of statistical analysis of data shown in Figure 6.17 it was found that, in section A, 82.91% of discontinuities show a rough condition. In section B most of the discontinuities belong in the rough category. It means that this condition can lead to an increase of the shear strength of the discontinuities in this particular site. Figure 6.18 represents a histogram of the frequency distribution of discontinuity roughnesses in section B.
Stepped Undulating Curvature
Plannar Stepped Undulating Curvature
Plannar

Figure 6.15 Probability Density Histogram of Discontinuity Curvature in Section A

Figure 6.16 Frequency Distribution of Discontinuity Curvature in Section B

Figure 6.17 Probability Density Histogram of Discontinuity Roughnesses in Section A

Figure 6.18 Frequency Distribution of Discontinuity Roughnesses in Section B
6.9.2.8 Discontinuity persistence

The persistence of discontinuities was considered to lead to discontinuity terminations in sections A and B of the quarry. The results of statistical analysis show that most of the discontinuity persistences (69.19%) in section A belongs to category one (e.g. At another). In the case of section B, from the results of statistical analysis, it was found that more than 69% of discontinuity persistences belong to At another category. The histograms of frequency distribution of discontinuity persistences in Marulan Quarry are given in Figures 6.19 and 6.20. The results of the statistical analysis of field data is presented in Table 6.5.

<table>
<thead>
<tr>
<th>Categories</th>
<th>Section A</th>
<th></th>
<th>Section B</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>At another</td>
<td>274</td>
<td>69.19</td>
<td>287</td>
<td>69.83</td>
</tr>
<tr>
<td>In rock</td>
<td>34</td>
<td>9.52</td>
<td>42</td>
<td>10.22</td>
</tr>
<tr>
<td>Beyond exposures</td>
<td>76</td>
<td>21.23</td>
<td>82</td>
<td>19.95</td>
</tr>
<tr>
<td>Total Values</td>
<td>357</td>
<td>100</td>
<td>411</td>
<td>100</td>
</tr>
</tbody>
</table>

Based on the results, it can be concluded that, in this particular rock mass, there is a very compact net work of joint systems which have a great effect on the instability of slope faces.

6.9.2.9 Discontinuity spacing

The frequency distribution of discontinuity spacings in a limestone rock mass was considered and recorded during the scanline surveys. It was found, from the results of statistical analysis of measured data that, in section A of the quarry, more than 55.4% of discontinuities have spacings of less than 0.2 m. The results of statistical analysis in section B show that 72.75% of discontinuities have spacings of less than 0.2 m. Altogether 89.87% and 93.18% of discontinuities have spacings of less than 0.4 m in sections A and B respectively.
Figures 6.21 to 6.22 display the values of mean spacing and standard deviation together with the discontinuity frequency of spacing ($\lambda$) and the number of observed values for sections A and B. A total of 768 discontinuity spacing values measured from the limestone quarry was processed and represented in histogram form. In addition, the negative exponential probability density distribution is drawn on each histogram, using the $\lambda$ as mean discontinuity frequency for the population. The results of the statistical analysis of field data of discontinuity spacings is presented in Table 6.6.

### Table 6.6 Frequency distribution of discontinuity spacings in different parts of quarry.

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of Values</th>
<th>Mean</th>
<th>Std. Div</th>
<th>&lt;0.2 metre (%)</th>
<th>Over one metre (%)</th>
<th>Maximum concentration at</th>
<th>Mode (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>357</td>
<td>0.19</td>
<td>0.15</td>
<td>55.46</td>
<td>1</td>
<td>0.0 - 0.2</td>
<td>55.46</td>
</tr>
<tr>
<td>Section B</td>
<td>411</td>
<td>0.16</td>
<td>0.14</td>
<td>72.75</td>
<td>0.0</td>
<td>0.0 - 0.2</td>
<td>72.75</td>
</tr>
</tbody>
</table>

The probability density histogram of discontinuity spacing for the sum of 768 measured discontinuities from Marulan Quarry is given in Figure 6.23 together with the superimposed best fit curve. The correlation coefficient for the best fit curve is ($r^2 = 0.97$), which illustrates the random distribution of the discontinuities at this quarry. The field data and curve constant parameters for sections A and B of the quarry are summarised on Table 6.7.

### 6.9.2.10 Joint Compressive Strength

In order to measure the joint compressive strength (JCS) of the discontinuity surfaces in the limestone quarry, a Schmidt hammer of N type was used. Using Schmidt hammer is the simple and cheapest method for determining the rock quality and rock strength of the rocks. The results of measured JCS on the joint surfaces of the limestone is presented in Figures 6.25 and 6.25 for sections A and B respectively. For more detail refer to section 4.5.3 of chapter 4.
Figure 6.19 Frequency Distribution of Discontinuity Terminations in Section A

Figure 6.20 Probability Density Histogram of Discontinuity Terminations in Section B

Figure 6.21 Probability Density Histogram of Discontinuity Spacings in Section A

Figure 6.22 Frequency Distribution of Discontinuity Spacings in Section B
Figure 6.23 Probability Density Histogram of Discontinuity spacings in Marulan Quarry

Fitted negative exponential probability density distribution, 
X = 5M/m 
No = 768 
Mean = 0.17 
Std. DIV = 0.14 
a = 83.18 
-b = 6.04 

$R^2 = 0.97$
Table 6.7 Summary of field data and negative exponential curve parameters

<table>
<thead>
<tr>
<th>Sources of data</th>
<th>Total scanline length (L (m))</th>
<th>Number of discontinuities</th>
<th>Number of total scanline length</th>
<th>Theoretical curve parameter ((\lambda = n/L))</th>
<th>Best fit curve parameters</th>
<th>R^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>67.8</td>
<td>357</td>
<td>6.9</td>
<td>5.26</td>
<td>87.86</td>
<td>6.2</td>
</tr>
<tr>
<td>Section B</td>
<td>65.79</td>
<td>411</td>
<td>8.1</td>
<td>6.25</td>
<td>61.93</td>
<td>5.63</td>
</tr>
<tr>
<td>Total data</td>
<td>133.59</td>
<td>768</td>
<td>7.6</td>
<td>5.75</td>
<td>83.18</td>
<td>6.04</td>
</tr>
</tbody>
</table>

6.9.2.11 Rock quality designation (RQD)

Different values of RQD for the sections A and B were calculated from joint surveys and theoretical RQD* calculated from equation 5.21 are presented in Table 6.8. As is clear from Table 6.8, for this particular porphyry rock mass, RQD* can be calculated to within 9% of the measured RQD using only the average number of discontinuities per metre (\(\lambda\)).

Table 6.8 A comparison between measured and theoretical RQD in a limestone rock mass

<table>
<thead>
<tr>
<th>Source of data</th>
<th>Total scanline length (L (m))</th>
<th>Average discontinuity frequency ((\lambda))</th>
<th>Measured RQD %</th>
<th>Theoretical RQD* %</th>
<th>Differences in RQD values (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>67.8</td>
<td>5.26</td>
<td>83.3</td>
<td>90.27</td>
<td>+6.97</td>
</tr>
<tr>
<td>Section B</td>
<td>65.79</td>
<td>6.25</td>
<td>78.32</td>
<td>87.4</td>
<td>+9.08</td>
</tr>
<tr>
<td>Total</td>
<td>133.59</td>
<td>5.75</td>
<td>80.68</td>
<td>82.11</td>
<td>+1.43</td>
</tr>
</tbody>
</table>

By using the above factor, the quality of rock masses were examined and the results are given in Table 6.9. The quality description chart (Figure 5.44) enables a consideration of the quality (RQP) and also (RQR) of the limestone rock mass. Different values of the rock quality indices calculated from the available data for the limestone rock mass are given in Table 6.9.
Table 6.9 Different Values of (RQD), (RQP), and (RQR) for a limestone rock mass.

<table>
<thead>
<tr>
<th>Source of data</th>
<th>Average discontinuity frequency (l)</th>
<th>RQD</th>
<th>RQP</th>
<th>RQR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>5.26</td>
<td>83.3</td>
<td>Good - Fair</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.04 - 0.96%</td>
<td></td>
</tr>
<tr>
<td>Section B</td>
<td>6.25</td>
<td>78.32</td>
<td>Good - Fair</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.02 - 0.98</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>5.75</td>
<td>80.68</td>
<td>Good - Fair</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.05 - 0.95</td>
<td></td>
</tr>
</tbody>
</table>

It is obvious, from Table 6.9 that, there is an inverse relationship between (RQR) and the average number of fractures along a scanline (λ), for a given (RQD) value. It means that the small value of (RQR) represents a better description of rock quality. From Table 6.9, it can be concluded that the rock mass is of a double quality type with (RQP) values of 0.04 and 0.96 for “good” and “fair” qualities respectively with a (RQR) equal to 0.96 which represents the greatest risk in section A. The results of data analysis for section B show that the rock mass presents a (RQP) values of 0.02 and 0.98 for “good” and “fair” qualities respectively with a (RQR) equaling 0.98 which is rather high.

6.9.2.12 Discontinuity length

The effect of discontinuity lengths and current methods of measurement were discussed in the chapter 5 section 5.3.2.11. Based on previous discussions and the joint survey data, the frequency distribution of discontinuous lengths in different parts of the quarry was examined. It was found, from the statistical analysis of data in section A, that more than 7.84 % of discontinuities have a length of less than 0.5 metre and 72 % of discontinuities have a length of more than one metre. The results of statistical analysis of data from section B show that more than 8.25 % of discontinuities have a length of less than 0.5 metre, and that 70 % of discontinuities have a length of more than one metre. From the results of statistical analysis it can be concluded that the rock mass under examination is governed by the joint sets, with considerable values of joint lengths, which can lead to the sliding or instability of some parts of rock slope faces. The results of a stability assessment in this quarry support these points.
Figure 6.24 Frequency Distribution of JCS in Section A

Figure 6.25 Probability Density Histogram of JCS in Section B

Figure 6.26 Frequency Distribution of Discontinuity Lengths in Section A

Figure 6.27 Probability Density Histogram of Discontinuity Length in Section B
Probability density histograms of frequency distribution of discontinuity lengths for the quarry are given in Figures 6.26 and 6.27. The results of statistical analysis of field data from different parts of quarry are presented in Table 6.10.

Table 6.10 Results of frequency distribution of discontinuity lengths in Sections A and B

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of Values</th>
<th>Total length of scanline L (m)</th>
<th>Mean (m)</th>
<th>Std. Div</th>
<th>&lt;0.5 m (%)</th>
<th>Over 3 m (%)</th>
<th>Maximum concentration at</th>
<th>Mode (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>357</td>
<td>67.8</td>
<td>1.58</td>
<td>0.97</td>
<td>7.84</td>
<td>22.41</td>
<td>3 - 3.5</td>
<td>21.85</td>
</tr>
<tr>
<td>Section B</td>
<td>411</td>
<td>65.79</td>
<td>1.56</td>
<td>0.98</td>
<td>8.27</td>
<td>22.40</td>
<td>3 - 3.5</td>
<td>22.14</td>
</tr>
<tr>
<td>Total</td>
<td>768</td>
<td>133.59</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

6.10 ANALYTICAL STATISTICS

6.10.1 Factor Analysis and Data Acquisition

The goal of factor analysis is to identify the not-directly observable factors based on a set of observable variables. It should be noted that by using some of the 768 discontinuity data from the limestone quarry (Marulan Quarry) the same result was obtained. It is clear that, the most important data parameters for defining the difference between rock masses and intact rock are the orientation of discontinuities, which plays a significant role in the stability of open pit mines.

6.11 APPRAISAL OF FACE STABILITY AT A LIMESTONE QUARRY IN MARULAN

In order to assess the stability of slope faces, the data collected from the joint surveys were analysed using statistical analysis methods to find the relationship between the discontinuity parameters. The data from discontinuity orientations were used for the
assessment of the potential mode of failures in two different parts of the limestone quarry, as described in the following sections.

6.11.1 Stability Assessment of the Slope Face in Section A

Section A is located in the northern part of the quarry and contains six benches. A discontinuities survey program was carried out in order to assess the stability of slope faces in this part of the quarry. Samples of naturally jointed rocks were collected during the site investigation. From direct shear tests a friction angle of 18.9 degrees and cohesion of 1.91 MPa were determined. The discontinuity dipping in this section shows an average of 70 - 80 degrees with high percentages of 32.77 %. A lower hemisphere equal area stereographic projection method was used for discontinuity data analysis. In the stability analysis a slope faces of 75 degrees with dip direction of 191 degrees were used to assess the potential mode of failures for this part of the quarry. All measurements were plotted on a stereographic net. Figure 6.28 shows pole and contoured pole plot nets in section A of the quarry.
Figure 6.28 Analysis of discontinuity orientation data of section A from Marulan Quarry
(a) Pole plot net     (b) Contoured plot net.
Figure 6.29 Analysis of pole plot data of section A from Marulan Quarry.

(a) Superimposed a polar net on the contoured pole plot net.  
(b) Stereographic projection of major discontinuity planes, slope face and friction angle.
According to the high density pole projection of 357 discontinuities measured from the slope face of bench six, five major joint sets were prominent, occurring at dip and dip directions of 17/267, 81/266, 78/297, 82/351 and 82/198 respectively. Figure 6.29 (a) represents a contour diagram superimposed on a polar net and Figure 6.25 (b) represents a stereographic projection of major discontinuity pales. It is clear from Figure 6.25 (b) that the intersecting lines $I_{45}$, $I_{65}$, $I_{63}$, and $I_{46}$ are situated within the shaded area, and represent the wedge failure potential in this section. It is recommended that the orientation of the slope face be changed to 68/214 in order to reduce potential instability.

6.11.2 Stability Assessment of the Slope Face in Section B

Section B is situated in the western section of the quarry. It contains some dolerite dykes intersecting the limestone rock mass which probably has an affect on the stability of the slope face in this part of the quarry (Figure 6.30). In this section a three dimensional joint survey program was carried out in order to assess the stability of slope faces and all discontinuities intersecting the scanline were recorded. The discontinuity characteristics such as discontinuity orientation, discontinuity dip angle, type of infilling materials, spacing, aperture, water condition, roughness, waveness, discontinuity length, and joint compressive strength (JCS) were measured and recorded in accordance with ISRM (1981) suggestions.

The present investigation was undertaken, as part of a program of slope failure investigations, in order to assess the overall stability of quarry. In section B of the quarry, the rock mass has been subjected to a small-scale toppling failure which resulted from discontinuity and slope face orientations (Figure 6.31 ). Some 411 discontinuity measurements were taken in three different directions of the limestone quarry in order to obtain reasonable information relating to the geological and geotechnical parameters affecting the stability of hard rock slopes. Figure 6.30 shows the pole plot (a) and contour plots (b) of all measured discontinuities.
Figure 6.30 Discontinuity orientation data analysis of section B from Marulan Quarry.
(a) Pole plot net        (b) Contoured pole plot net
Figure 6.31 Analysis of cluster poles of section B from Marulan Quarry. (a) A polar net superimposed on the contoured pole plot net  (b) Stereographic projection of dominant discontinuity planes, slope face and friction cone.
Results of the statistical analysis of discontinuity data shows that the percentages of discontinuity dipping gives a high percent of (34.55%) 80 - 90 degrees in section B. It indicates that the joints in this section are sharply inclined in their dip.

A lower hemisphere equal area stereographic projection method was used to assess the stability of slope faces in this part of the quarry. In the stability analysis, a slope face of 75 degrees with a dip direction of 111 degrees and a friction angle of 18.9 degrees, derived from direct shear tests of naturally jointed samples, were used to assess the potential mode of failures. All measurements were plotted on a stereographic net.

Based on the high density pole projection of 411 discontinuities measured from the slope face of this part of the quarry, four major joint sets were prominent; occurring at dip and dip directions of 51/022, 81/205, 82/360, and 35/208 respectively. From Figure 6.29 (b) is clear that the intersecting lines $I_{15}, I_{23}, I_{14}, I_{35}$, and $I_{13}$ are situated within the shaded area and present a wedge failure potential in this section. Figure 6.29 (a) represents, superimposed, a polar net and stereographic projection of major discontinuity pales respectively. The potential of toppling is possible between planes one and four and also a potential of sliding is predicted in the direction of intersecting lines $I_{24}$. In order to prevent the potential of instability, or reduce risk, it is recommended that the orientation of slope face be changed to 70/140 or that the slope angle be reduced to 65 degrees. It should be noted that, in this condition, the potential of failure by the intersection lines $I_{23}, I_{25}$, and $I_{35}$ can be disallowed.

6.12 EVALUATION OF ROCK MASS STRENGTH PROPERTIES

Naturally jointed samples from the limestone rock mass were collected from the working slopes in order to evaluate the shear strength parameters of the rock mass. Following the direct shear tests (described in chapter 4), shear strength parameters were obtained. As shown in Table 4.3, the results give an average friction angle of 18.9 degrees and cohesion of 1.912 MPa for sections A and B. The orientations of dominant joint sets are given in Table 6.11.
Table 6.11 Orientations of Major Discontinuity Planes and Slope Faces

<table>
<thead>
<tr>
<th>Structural region (Dip - dip direction)</th>
<th>Section A</th>
<th>Section B</th>
</tr>
</thead>
<tbody>
<tr>
<td>First predominant joint set</td>
<td>17 / 267</td>
<td>51 / 022</td>
</tr>
<tr>
<td>Second predominant joint set</td>
<td>81 / 266</td>
<td>81 / 205</td>
</tr>
<tr>
<td>Third predominant joint set</td>
<td>78 / 297</td>
<td>82 / 360</td>
</tr>
<tr>
<td>Fourth predominant joint set</td>
<td>82 / 351</td>
<td>35 / 208</td>
</tr>
<tr>
<td>Fifth predominant joint set</td>
<td>82 / 198</td>
<td>—</td>
</tr>
<tr>
<td>Slope face</td>
<td>75 / 191</td>
<td>75 / 111</td>
</tr>
<tr>
<td>Changed slope face</td>
<td>68 / 214</td>
<td>70 / 140</td>
</tr>
</tbody>
</table>

6.13 ROCK MASS CHARACTERISATION AT MARULAN QUARRY

For the characterizing of the rock mass at Marulan Quarry the following distinct steps were carried out:

- Determining engineering properties of intact rock by laboratory testing
- Estimating rock mass strength from the laboratory and field data
- Classification of rock mass using
  - RMR system
  - Q System
  - Weakening Coefficient system

6.13.1 Engineering Properties of Intact Rock

The method and procedures of rock engineering properties considering have been described in detail in Chapter 3. Only the most important properties of intact rock limestone samples achieved from laboratory tests are presented in Table 6.12. It should be noted that these engineering properties were used as input data in order to establish the rock mass strength and for rock mass classification.
Table 6.12 Engineering properties of intact rock samples from Marulan Quarry

<table>
<thead>
<tr>
<th>Engineering properties of intact rock</th>
<th>Structural regions A and B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial compressive strength (MPa)</td>
<td>77.84</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>3.81</td>
</tr>
<tr>
<td>UCS derived from diametral point load test (MPa)</td>
<td>77.37</td>
</tr>
<tr>
<td>UCS derived from axial point load (MPa)</td>
<td>86.13</td>
</tr>
<tr>
<td>Poisson’s ratio (MPa)</td>
<td>0.35</td>
</tr>
<tr>
<td>Elastic modulus (GPa)</td>
<td>18.45</td>
</tr>
<tr>
<td>Friction angle</td>
<td>45.10</td>
</tr>
<tr>
<td>Bulk density kg/m³</td>
<td>2660.06</td>
</tr>
</tbody>
</table>

6.13.2 Evaluation of the Rock Mass Strength Parameters

In order to estimate the rock mass strength parameters for the limestone rock mass at the Marulan Quarry, some rock blocks containing natural joint samples from the slope faces of the quarry were collected and tested using a direct shear box. The detail of methodology of preparation and testing of the samples containing natural joints was explained in the chapter 4. From the results of direct shear tests carried out on naturally jointed samples, the value of the internal friction angle was found to be 18.88 degrees for sections A and B of the quarry and the value of the cohesion was determined 1.912 Mpa for these sections. From the rock mass classification (RMR) the values of cohesion and friction angle of rock mass were found to be 200 - 300 kPa and 25° - 30° for section A and <100 kPa and <15° for section B respectively.

6.13.3 Application of Rock Mass Classification Systems

For the purpose of this research, some currently used rock mass classifications such as the geomechanics classification (RMR) and Norwegian Geotechnical Institute (NGI) system or Q Index, as well as the Weakening Coefficient system (WC) were examined. The application of these classifications are described in the following sections.
6.13.3.1 Application of geomechanics classification system (RMR)

The geotechnical parameters contributing to the RMR system were determined and the rating of each parameter were obtained from the section A of Table 2.10. The calculated RMR rating values for Section A and B of the limestone quarry are given in Table 6.13. According to the results of rock mass classification, the rock mass quality in section A is fair and in section B is a very poor class of rock mass.

<table>
<thead>
<tr>
<th>Rock Mass Parameters</th>
<th>Section A</th>
<th>Section B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value</td>
<td>Rating</td>
</tr>
<tr>
<td>U. C. S. (MPa)</td>
<td>77.84</td>
<td>7</td>
</tr>
<tr>
<td>RQD %</td>
<td>83.3</td>
<td>17</td>
</tr>
<tr>
<td>Discontinuity Spacing (m)</td>
<td>0.19</td>
<td>8</td>
</tr>
<tr>
<td>Discontinuity Condition</td>
<td>Class III</td>
<td>20</td>
</tr>
<tr>
<td>Ground Water Condition</td>
<td>Dry</td>
<td>15</td>
</tr>
<tr>
<td>Orientation Rating</td>
<td>Class III</td>
<td>25</td>
</tr>
<tr>
<td>Total RMR Rating</td>
<td>57</td>
<td></td>
</tr>
</tbody>
</table>

6.13.3.2 Application of Q system

The Q system was examined in the Marulan Quarry in order to quantitatively evaluate the quality of the limestone rock mass. This system is based on six geotechnical parameters as it was explained in detail in chapter 2 of this thesis. This system is based on six geotechnical parameters. Some of them have a direct and positive relationship with the quality of the rock mass and some have a reverse relationship with the quality of the rock mass. The Q index can be calculated by the following equation:
This system of classification is mainly used in tunneling and underground excavations but is also useful for ascertaining the rock mass strength and quality of rock masses in open pit mining. It is obvious that the values obtained by rock mass classification are not 100% accurate and insure, but they can give an overall view of the behaviour of the rock mass. The results of the Q system of classification for the limestone rock mass is given in Table 6.14.

Table 6.14 Engineering Rock Mass Classification (Q System) of the Marulan Quarry.

<table>
<thead>
<tr>
<th>Rock Mass Parameters</th>
<th>Structural region (A)</th>
<th>Structural region (B)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock Quality Designation (RQD) %</td>
<td>Value 83.3</td>
<td>Rating 83.3</td>
</tr>
<tr>
<td>Joint Set Number (Jn)</td>
<td>5</td>
<td>15</td>
</tr>
<tr>
<td>Shear Strength Factor (Jr / Jn)</td>
<td>3/2</td>
<td>1.5</td>
</tr>
<tr>
<td>Joint Water Reduction Factor (Jw)</td>
<td>Dry</td>
<td>1</td>
</tr>
<tr>
<td>Rating Adjustment Factor (RAF)</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Total rating</td>
<td>1.65</td>
<td></td>
</tr>
<tr>
<td>Description</td>
<td>Poor</td>
<td></td>
</tr>
</tbody>
</table>

6.13.3.3 Application of weakening coefficient classification system (WC)

The Weakening Coefficient classification system (WC) is used in the design of rock slopes and was introduced by Singh (1986). The most important discontinuity parameters contributing to this system are; RQD, joint spacing, joint surface index, joint filling materials and, finally, the discontinuity aperture. The weakening coefficient values of the limestone rock mass were quantified according to the discontinuity parameters and
the quantified values are presented in Table 6.15. It may be recalled that the relationship between these discontinuity parameters can be represented by the following equation;

\[ K = K_1 \times K_2 \times K_3 \times K_4 \]  

(6.3)

where;

- \( K_1 \) = discontinuity aperture
- \( K_2 \) = discontinuity spacing
- \( K_3 \) = joint surface index
- \( K_4 \) = joint filling index

The Weakening Coefficient can be calculated by the following equation

\[ WC = R \times K \]  

(6.4)

where

- \( R = RQD \)

Table 6.15 The Results of Weakening Coefficient Classification of Limestone Rock in Marulan Quarry

<table>
<thead>
<tr>
<th>Rock Mass Parameters</th>
<th>Section A</th>
<th></th>
<th>Section B</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value</td>
<td>Rating</td>
<td>Value</td>
<td>Rating</td>
</tr>
<tr>
<td>RQD %</td>
<td>83.3</td>
<td>0.83</td>
<td>78.32</td>
<td>0.78</td>
</tr>
<tr>
<td>Discontinuity Spacing (m)</td>
<td>0.19</td>
<td>0.7</td>
<td>0.16</td>
<td>0.7</td>
</tr>
<tr>
<td>Joint Surface</td>
<td>Rough</td>
<td>0.9</td>
<td>Rough</td>
<td>0.9</td>
</tr>
<tr>
<td>Joint Infilling</td>
<td>Open &lt;5 mm</td>
<td>0.7</td>
<td>Open &gt;5 mm</td>
<td>0.6</td>
</tr>
<tr>
<td>Discontinuity Aperture</td>
<td>2 - 6 mm</td>
<td>0.7</td>
<td>2 - 6 mm</td>
<td>0.7</td>
</tr>
<tr>
<td>Weakening Coefficient (WC)</td>
<td>0.256</td>
<td></td>
<td>0.206</td>
<td></td>
</tr>
<tr>
<td>Orientation Rating</td>
<td>0.29 \times 0.37</td>
<td>0.13 \times 0.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Weakening Coefficient (WC)</td>
<td>0.11</td>
<td></td>
<td>0.013</td>
<td></td>
</tr>
<tr>
<td>Description</td>
<td>Moderate</td>
<td></td>
<td>Poor</td>
<td></td>
</tr>
</tbody>
</table>
A study of strength and deformation measurements for the limestone rock mass, along with consideration of the effect of fracturing using rock mass classification and statistical analysis of discontinuity data provided a range of valuable information for the aim of engineering design of slopes in limestone rock mass at the Marualn Quarry.

Based on the laboratory testing, field investigation programs and using different rock mass classification, a summary of pertinent physical characteristics of limestone and also most important engineering properties of limestone rock mass are given in Table 6.16.

Table 6.16 Engineering description of limestone in Marulan quarry.

<table>
<thead>
<tr>
<th>Property</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>Recrystallised and medium grained, light to dark gray limestone</td>
</tr>
<tr>
<td>Rock Mass</td>
<td>Typically thin to thickly bedded and semi-weathered rock mass</td>
</tr>
<tr>
<td>Rock Quality Designation (RQD)</td>
<td>83.8 % in section A and 78.32 % in section B</td>
</tr>
<tr>
<td>Joint set numbers and orientations</td>
<td>Five in section A and Four in section B, with vertical and horizontal orientations</td>
</tr>
<tr>
<td>Joint roughness</td>
<td>Rough to smooth in sections A and B</td>
</tr>
<tr>
<td>Water condition</td>
<td>May dry rock mass</td>
</tr>
<tr>
<td>Rock mass rating and quality</td>
<td>12 - 57 (fair in section A and very poor in section B)</td>
</tr>
<tr>
<td>Cohesion and friction angle of the rock mass</td>
<td>Cohesion of 200 - 300 kPa for section A and &lt;100 kPa for section B. Internal friction angle of 25 - 35° for section A and &lt;15° for section B.</td>
</tr>
</tbody>
</table>
6.15 CONCLUSIONS

Results of the stability assessment in Marulan Quarry showed that five major joint sets in section A and four dominant discontinuity sets in section B are affecting the stability of slope faces of the quarry. It was also identified that there is a potential of some wedge and toppling failures in the sections A and B of the quarry. Due to the presence of these major joint sets and also the presence of some dolerite dykes in different parts of the quarry, changing the slope faces in these parts was recommended as part of remedial measures.

The frequency distribution of the discontinuity data for the most important of discontinuity parameters was achieved by using statistical analysis techniques and the negative exponential probability density distribution model was proposed for the frequency of distribution of discontinuity spacings in different parts of the quarry. Results of statistical analysis showed that there is a significant relationship between discontinuity parameters acting on the stability of hard rock slopes in Marulan Quarry. It was also established for the limestone rock mass at this quarry that, the length of discontinuity is the most important factor affecting hard rock slope stability.

Results of different rock mass classifications carried out in Marulan Quarry showed the limestone rock mass is a moderate to very poor quality in sections A and B respectively. The value of the cohesion for sections A and B was found to be 200 - 300 kPa in section A and <100 kPa for section B and value of internal friction angle was found to be 25° - 30° for section A and <15° in sections B respectively.
Chapter 7
Application of Modified Rock Mass Classification Systems on Stability of Basalt Quarry
7.1 INTRODUCTION

This chapter presents, an assessment of the geotechnical parameters influencing the instability of slope faces of a hard rock quarry (basalt) at Dunmore Quarry. The method of investigation consisted of a three dimensional geological mapping of the discontinuities from the slope faces of the quarry and a laboratory testing program of intact rock and naturally jointed samples. A statistical analysis of the discontinuity data measured from the joint survey program was also carried out which resulted in the developing correlations between various parameters defining the rock mass characteristics. This chapter presents an application of stereographic projection techniques for the stability assessment of the slope faces at Dunmore quarry.

7.2 DUNMORE QUARRY: GENERAL INFORMATION

The existing quarry is located 1.8 Km off the western side of the Princes Highway, in the Dunmore region midway between Kiama and Albion park, approximately 110 kilometres
south of Sydney on the south coast of New South Wales. The location of the study area is presented in Figure 7.1.

The area has a rainy climate with cool summers and mild winters with an average annual rainfall of 1100 mm. The average daily temperature over the year is 17.5 degrees C. In the summers the dominant winds are generally from the south or northeast, while in the winters they are from the west and southwest (Bowman, 1974).

Access to the area is via the Princes Highway and adjoining sealed Swamp and Rocklow roads. A general view of Dunmore Quarry is presented in Figure 7.2. In this quarry, extraction of stone is carried out by drilling and blasting. Quarry spalls are loaded by a front end loader into 50 tonne capacity haul trucks, for transport to crushing and screening facilities. Production from the quarry is stored, in a stockpile area, to await road or rail transport to market destinations.

A spur of the Illawarra rail line at Shellharbour Station services the quarry. In Dunmore Quarry, quarrying and crushing activities occur between the hours of 6.00 am to 6.00 pm Monday to Friday, and 6.00 am to 12.00 noon on Saturdays.

Blasting fractures and dislodges rock fragments from the rock face into manageable sizes for transportation and handling by the processing plant. To achieve this, blast holes are drilled into the rock face in a designed pattern with particular attention being given to their angle, depth, and spacing. The holes are filled with explosive charges, usually consisting of a small amount of high explosive followed by a bulk explosive which makes up the remainder of the charge. Quarry blasting is scheduled to take place approximately once a month. Dunmore quarry currently provides rock for a range of products, including various sizes of concrete and cover aggregate, prepared road base and fine crushed rock, rail ballast, gabion, asphalt aggregate and break water material.
Figure 7.1 Locality map of the study area (Modified after Bowman, 1974).
Figure 7.2 General view of the Dunmore Quarry
The transportation system in Dunmore quarry includes two systems - road and railway, and according to the current sales demand, approximately 2000 tonnes of product leaves the quarry on a daily basis. This translates to approximately 100 daily truck movements. In road transportation, loading is by front end loader in the stockpile area and trucks leave the site after weighing and sales. The loading system in Dunmore Quarry is shown in Figure 7.3. The vertical joint systems (columnar structure) in the quarry is presented in Figure 7.4. It should be noted that this geological structure will have a considerable effect on the stability of slope faces and also in the efficiency of the blasting.

7.3 PREVIOUS WORKS

Due to the significance of the Illawarra Coal resources, the Southern Sydney Basin has been the site of considerable activity by many researchers. The first work was carried out by (Jaquet et al, 1905), and (Harper, 1915), quoted by (Gibbons, 1992).

The quarrying industry began to have importance in the Kiama by 1870 and in 1921 the Dunmore Quarry was established as the first large quarry outside the Kiama center. Latite rock mass was identified in the Croome Frame area by BMG Quarries Ltd. A limited amount of exploratory drilling was carried out on the site between 1970 and 1990. During the preliminary investigations, data from drilling showed the significant presence of high quality latite resources in the area. The Bumbo Latite Member is the geological unit in which Dunmore Quarry is worked. Latite is a fine grain, extrusive, igneous rock, similar to basalt, in appearance and mechanical properties but with a higher percentage of combined silica. It should be noted that exposed latite and sandstone are easily visible in the Kiama area. Dunmore Quarry has supplied the Illawarra, Wollongong and Sydney market for more than 60 years. The quarry has had many extensions during its existence, and its most recent major extension took place in 1986.
Figure 7.3  Illustration of the loading system in Dunmore Quarry

Figure 7.4  Illustration of a typical geological structures (columnar joint systems) in Dunmore Quarry
A Development Application (DA), with accompanying Environmental Impact Statement (EIS), was lodged with Shellharbour Council for the continuation of quarrying into an area owned by the NSW State Rail Authority (SRA). The progression of quarry development has been from east to west over the past 60 years. Extraction in the current quarry is progressively advancing toward the Coorme Frame area.

7.4 GENERAL GEOLOGY IN DUNMORE-KIAMA

The Sydney basin contains a sequence of sedimentary and volcanic rocks which covers an area of about 36,000 km onshore and 16,000 km offshore. The maximum thickness of this sequence has been measured in the northern part of the basin where up to 5900 m of marine and non-marine rocks have been recorded (Scheibnerova, 1982). The Sydney Basin is the southern most part of the Sydney-Bowen Basin. It is a structural basin which extends from Townsville in Queensland down to the Batemans Bay region of New South Wales. The structural boundaries of the Sydney Basin, developed over a long period of time, concluding in the Late Triassic period (Bembrick et al, 1980).

The boundaries of the Sydney Basin are both structural and depositional. To the northwest, west and south, boundaries are depositional, lying on the Paleozoic rocks of the Lachlan Fold Belt. The basin is bounded to the northeast by the Hunter-Mooki thrust fault system. This system is thought to have developed during stabilisation of the New England block in association with the Hunter Orogeny (Bembrick et al, 1980) quoted by (Kay, 1985).

7.5 GEOLOGY AND SITE INVESTIGATION IN DUNMORE QUARRY

The Dunmore quarry is located in the southern Sydney Basin. Deposition in the southern Sydney Basin commenced in the Late Carboniferous and continued through the Permian to at least the late Triassic. (Carr, 1982) stated that the Broughton Formation is a portion of the Gerringong Volcanic Facies within the Shoalhaven Group.
The members of this formation are the Westley Park sandstone (Basal unit), Blow Hole Latite, Kiama Sandstones, Bumbo Latite, and Jambewarra Latite (Gibbons, 1992). It is believed that the Broughton Formation has been formed during a marine regression and the environment of the deposition of the Shoalhaven Group can broadly be seen as shallow marine deposits.

The term Latite has been defined as an extrusive intermediate igneous rock composed of approximately equal amounts of alkali feldspar and sodic plagioclase (Gibbons, 1992). The Bumbo Latite Member (150 m thickness) is stratigraphically overlain by the Jamberoo sandstone member (60 m maximum thickness) and underlie by the Kiama Sandstone (76 m thickness) as presented in Figure 7.5.

The Bumbo Latite Member has been recognised as a succession of three lava flows interbedded with marine sediments (Bowman, 1974) quoted by (Gibbons, 1992). The three lava flows are the Lower Flow (F1), Middle Flow (F2), and Upper Flow (F3) respectively. According to the results of a diamond drilling program (Robertson, 1974), lying between the F1 and F2 is an extensive zone of brecciation and the boundary between F2 and F3 is marked by an alteration zone. It should be noted that the contact zones between three lava flows are recognised in Dunmore Quarry (Robertson, 1974).

In Dunmore Quarry, the Bumbo Latite shows extraordinary columnar jointing. The diameter of columns are between one to more than two meters as shown in Figure 7.6. The top flow consists of glassy plagioclase crystals up to 2.5 centimetre or more in length and breadth in a dark gray, almost black groundmass. The plagioclase can be easily identified by the beautiful multiple twinning.

Kiama is situated on the red coloured Kiama Tuffs. Good exposures of these occur in road cuttings and in the cliff sections. They can be seen in the cliff and rock platform near the swimming pool. The tuffs could best be described as tuffaceous sandstone and their colour is due to a ferruginous cement. They are well bedded with well defined conglomeratic bands being common (Nashar, 1967).
Figure 7.5 Stratigraphy column of the Bumbo Latite Member.
7.6 STRUCTURAL GEOLOGY

There is limited information available on the structures contained within the Broughton Formation. The structural geology recorded for the Southern Sydney Basin is concerned with coal bearing sediments. The regional structure is a large shallow basin open and plunging at 1° - 2° to the east.

A localized dip of up to 12° dip occurs towards the center of the basin. The axis of the trough appears to pass through the Minnamurra River crossing on the Albion Park - Jamberoo Road, and then moves slightly north - northeast through Minnamurra Township (Gibbons, 1992). Figure 7.7 shows a part of high wall benches in Section A of the quarry. It is clear from the Figure 7.7 that, this part of the quarry is unstable and based on the field observation a local wedge failure has been occurred in this area during the joint survey program.

7.7 STATISTICAL ANALYSIS OF DISCONTINUITY DATA IN DUNMORE QUARRY

7.7.1 INTRODUCTION

In the present chapter, a statistical analysis of field data was carried out in order to consider the importance of discontinuity parameters and their effects on the stability of slope faces in a basalt rock mass.
Figure 7.6  Illustration of a typical columnar latite (cross section) in Dunmore quarry, the Clar compass is scale on the floor.

Figure 7.7  Representation of unstable high wall bench in section A of Dunmore Quarry.
7.8 METHODOLOGY OF DISCONTINUITY DATA COLLECTION AND ANALYSIS

Methods suggested by the International Society for Rock Mechanics (ISRM, 1981) were used for the description of individual rock mass discontinuities. In order to study the effect of discontinuity characteristics on the stability of slope faces in open pit mines the quarry was divided into A and B sections and a three dimensional geological mapping was carried out on the exposed rock on the slope faces of Dunmore Quarry. It should be noted that section A occupies the eastern part of the quarry and section B occupies the southern part of the quarry.

The joint survey program was carried out using vertical and horizontal scan-lines. Some 872 readings were taken from the discontinuities which intersected the scan-line. The discontinuity data collected were used for statistical analysis, rock mass classification and slope stability evaluation, in order to identify the effect of discontinuities on slope stability, as well as to define the modes of failure and to provide information for remedial methods.

7.9 DESCRIPTIVE STATISTICS

For the purpose of the statistical analysis of discontinuity data obtained from the geological mapping, both descriptive statistics and analytical statistics techniques were examined. In this regard, descriptive statistical analysis was carried out on discontinuity data collected from joint survey program. In the following sections, the frequency distribution of most important of discontinuity characteristics are described.

7.9.1 Discontinuity Dip Angle

The dip angle of discontinuities is a significant factor affecting the stability of slope faces in open pit mining operations. This factor also has a great effect on the cost of operation during the life of an open pit mine. Consequently, the frequency distribution of dip angle
was examined at the Dunmore Quarry. The statistical analysis results are given in Table 7.1. As shown in this table, more than 69% of all discontinuities have a dip angle of over 70 degrees in section A. Altogether, more than 75% of discontinuity dip angles are over 60 degrees.

Table 7.1 Representation of Probability Density Distribution of Discontinuity Dip Angles in Dunmore Quarry

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of values</th>
<th>Mean</th>
<th>Std. Div</th>
<th>Minimum (degrees)</th>
<th>Maximum concentration at</th>
<th>Mode (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>450</td>
<td>68.05</td>
<td>23.73</td>
<td>5</td>
<td>80 - 90</td>
<td>52.89</td>
</tr>
<tr>
<td>Section B</td>
<td>422</td>
<td>65.06</td>
<td>28.37</td>
<td>5</td>
<td>80 - 90</td>
<td>56.06</td>
</tr>
</tbody>
</table>

The results of statistical analysis in section B showed that more than 66% of all discontinuities have a dip angle of over 70 degrees and more than 70% of dip angles are over 60 degrees (Table 7.1). These high values of the dip angle can influence the stability of the slope faces in hard rock quarry operations. It should be noted that, based on field observations some of the discontinuities have a dip angle of more than 85 degrees in this Quarry. Depending upon the slope face orientations in different parts of the quarry and some of these major discontinuities are parallel to the slope faces and some are normal to the slope faces. A graphical presentation of the frequency distribution of dip angles in the sections A and B are presented in Figures 7.8 and 7.9.

7.9.2 Discontinuity Aperture

Discontinuity aperture affects the stability of hard rock slopes particularly, when, it is filled by soft and fine grain materials such as clay or decomposed materials (Chlorite). This parameter is normally measured as the perpendicular distance between discontinuity walls. The results of statistical analysis from sections A and B are presented in Figures 7.10 and 7.11.
Figure 7.8 Probability Density Histogram of Discontinuity Dip Angle in Section A

Figure 7.9 Frequency Distribution of Discontinuity Dip Angle in Section B

Figure 7.10 Frequency Distribution of Discontinuity Aperture in Section A

Figure 7.11 Probability Distribution of Discontinuity Aperture in Section B
These results showed that more than 62% of discontinuities have an aperture greater than 2 mm in section A and around 64% of discontinuities have an aperture of more than 2 mm in section B. From the statistical results it can be concluded that the rock mass has been affected by the tectonical activities during previous geological periods. It should be noted that when the width of the opening between the discontinuity walls represents a considerable value this factor can have a great influence on the stability of slope faces of hard rocks in open pit mining.

7.9.3 Orientation of Discontinuities

Since the orientation of discontinuities is an important factor affecting the stability of hard rock slopes, a statistical examination of the frequency distribution of discontinuity orientations was carried out on the discontinuities from Dunmore Quarry. Table 7.2 shows a frequency distribution of the discontinuity orientations in sections A and B of the Dunmore Quarry. Graphical presentation (Rose diagrams) of frequency distribution of discontinuity orientations in sections A and B of the Dunmore Quarry are presented in Figures 7.12 and 7.13.

From the results of statistical analysis it was found that the maximum concentration of discontinuity orientations are situated between 280 - 300 degrees in section A. The Mean value of discontinuity orientations was measured 228.63 degrees. In section B, the results of statistical analysis showed that the maximum concentration of discontinuity orientations are situated between 200 - 220 degrees with a high percentage of 15.64%.

Table 7.2 Results of Descriptive Statistical Analysis of Discontinuity Orientations

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of values</th>
<th>Mean</th>
<th>Std. Div</th>
<th>Minimum (degrees)</th>
<th>Maximum concentration at</th>
<th>Mode (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>450</td>
<td>228.63</td>
<td>62.89</td>
<td>90</td>
<td>200 - 220</td>
<td>13.78</td>
</tr>
<tr>
<td>Section B</td>
<td>422</td>
<td>183.76</td>
<td>56.05</td>
<td>100</td>
<td>120 - 140</td>
<td>14.93</td>
</tr>
</tbody>
</table>
7.9.4 Water Condition

Effect of pore water pressure in soft rocks and water flowing through discontinuities in hard rocks were examined in Dunmore Quarry by visual examination. Since this factor may have considerable influence on the stability of the pit slope faces. Thus, regular examination of the hydrogeological condition of the rock face in the design and extraction phases of open pit mining should be a regular part of routine examination. According to current reports and field observations there is no evidence of water trace or seepage in this quarry. Nearly all discontinuities are in a dry condition in sections A and B of Dunmore Quarry. The results of statistical analysis of field data are presented in Table 7.3.

Table 7.3 Results of statistical analysis of discontinuity water condition in sections A and B of the basalt rock mass.

<table>
<thead>
<tr>
<th>Categories</th>
<th>Section A</th>
<th></th>
<th>Section B</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>Dry</td>
<td>450</td>
<td>100</td>
<td>419</td>
<td>99.29</td>
</tr>
<tr>
<td>Seepage</td>
<td>0.0</td>
<td>0.0</td>
<td>3</td>
<td>0.71</td>
</tr>
<tr>
<td>&lt;0.1 l/sec</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Total Values</td>
<td>450</td>
<td>100</td>
<td>422</td>
<td>100</td>
</tr>
</tbody>
</table>

7.9.5 Discontinuity Curvature

Discontinuity curvature or waviness are other parameters which can have an effect on the stability of slope faces in the Dunmore quarry was examined at the Dunmore Quarry quantitatively in accordance with ISRM (1981) suggestions. A graphical presentation of the probability density histograms of the discontinuity curvatures in sections A and B are presented in Figures 7.14 and 7.15.
Figure 7.12 Rose Diagram showing the direction of 450 Joint traces in the Section A

Figure 7.13 Rose Diagram Showing Orientation of 422 Joint Traces in Section B

Figure 7.14 Probability Density Histogram of Discontinuity Curvature in Section A

Figure 7.15 Frequency Distribution of Discontinuity Curvature in Section B
The results of statistical analysis are given in Table 7.4 for sections A and B. The results obtained from statistical analysis showed that the frequency of the different types of curvatures in this quarry nearly show a balance between the different categories.

Table 7.4 Results of statistical analysis of discontinuity curvatures in a basalt quarry.

<table>
<thead>
<tr>
<th>Categories</th>
<th>Section A</th>
<th></th>
<th>Section B</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>Stepped</td>
<td>139</td>
<td>30.89</td>
<td>139</td>
<td>32.94</td>
</tr>
<tr>
<td>Undulating</td>
<td>126</td>
<td>28</td>
<td>176</td>
<td>41.7</td>
</tr>
<tr>
<td>Planar</td>
<td>185</td>
<td>41.11</td>
<td>107</td>
<td>25.35</td>
</tr>
<tr>
<td>Total Values</td>
<td>450</td>
<td>100</td>
<td>422</td>
<td>100</td>
</tr>
</tbody>
</table>

7.9.6 Discontinuity Infilling Materials

As the shear strength parameters of the rock mass are controlled by the characteristics of infilling materials of discontinuities the field investigations at Dunmore quarry examined the following features.

- Nature of infilling material by visual examination whether sandy or clayey material.
- State of filling material by examining the degree of alteration.
- Type of infilling material determined by X-RD method

In addition, samples of infilling materials from different parts of the Dunmore Quarry were collected and tested using the X-RD method and the results are given in Appendix B. Figures 7.16 and 7.17 show a considerable thickness of infilling materials and staining of the discontinuity planes in section A of the quarry. Figures 7.18 and 7.19 show histograms of frequency distribution of discontinuity infilling materials in sections A and B.
According to the statistical analysis of the frequency distribution of infilling materials in section A, the most important categories of infilling materials are clear (No filled) and calcite types with high percentages: 48.44% and 20.89% respectively. Statistical results in section B showed that around 35% of discontinuity openings are clear (No filled) and more than 32% of them showed Staining.

7.9.7 Discontinuity Roughness

In this survey at Dunmore Quarry, the small scale roughness of discontinuity surfaces were examined by comparison of each 10 cm length of the joint surfaces with the standard profiles suggested by ISRM (1981). Figures 7.20 and 7.21 present the frequency distribution of roughness of discontinuities in sections A and B of Dunmore Quarry. The results of the statistical analysis of the large scale discontinuity roughness in section A showed that more than 76.44% of discontinuities belong to the first category (rough) and in section B around 87% of discontinuity roughness belongs to the rough category. This will increase the shear strength parameters.

7.9.8 Discontinuity Persistence

The persistence of the discontinuities has a significant effect on the shear resistance and stability of rock slopes. In practice it is very difficult to study the discontinuity persistence within the rock mass directly. There are some mathematical methods which enable us to define the discontinuity persistence type in a rock mass. In this chapter, termination of the discontinuities was observed in the joint survey program. Figures 7.22 and 7.23 present probability density histograms of discontinuity terminations in the quarry. The frequency distribution of discontinuity terminations in sections A and B is presented in Table 7.5

From the results of statistical analysis, it was found that most of the discontinuity persistence in sections A and B of the quarry have a termination of category one (At another) with 60.22% and 58.77% respectively.
Figure 7.16 Illustration of a considerable thickness of infilling materials in section A of Dunmore Quarry

Figure 7.17 Illustration of staining processes on the discontinuity walls in section A of Dunmore Quarry
Figure 7.18 Frequency Distribution of Discontinuity Infilling Materials in Section A

Figure 7.19 Probability Density Histogram of Discontinuity Infilling Materials in Section B

Figure 7.20 Probability Density Histogram of Discontinuity Roughness in Section A

Figure 7.21 Frequency Distribution of Discontinuity Roughness in Section B
This means that there is a net-work and continuous jointed system in these parts of the quarry which may lead to the instability of the slope faces in these parts of the quarry.

Table 7.5 Results of statistical analysis of discontinuity persistence in basalt quarry.

<table>
<thead>
<tr>
<th>Categories</th>
<th>Section A</th>
<th>Section B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>At another</td>
<td>271</td>
<td>60.22</td>
</tr>
<tr>
<td>In rock</td>
<td>53</td>
<td>11.78</td>
</tr>
<tr>
<td>Beyond exposures</td>
<td>126</td>
<td>28</td>
</tr>
<tr>
<td>Total Values</td>
<td>450</td>
<td>100</td>
</tr>
</tbody>
</table>

7.9.9 Discontinuity Length

The recognition and analysis of discontinuity characteristics is an important bridging link between the concepts of rock behaviour and the actual performance of rocks in the vicinity of structures in a rock mass (Farmer, 1983). The trace of discontinuity length is related to the persistence of a discontinuity, and hence is of geomechanical significance (Pahl, 1981). The details of the effect of discontinuity lengths and current methods of measuring were discussed in chapters 5 and 6. A frequency distribution of lengths of discontinuities in sections A and B of Dunmore Quarry is presented in Table 7.6. A graphical representation of discontinuity lengths is presented in Figures 7.24 to 7.25.

Results obtained from statistical analysis showed that more than 31% of discontinuities have a length of less than one metre in section A and 28% of discontinuities have a length of more than three metres. The results of the statistical analysis of data from section B showed that more than 29% of discontinuities are less than one metre in length while the maximum concentration of the discontinuity lengths (31.75%) belongs to discontinuities with a length of between 3 to 3.5 metres. From Table 7.6 it is clear that a discontinuity length with a considerable mean value can lead to failure or large scale sliding on the high wall of hard rocks in sections A and B of the quarry.
Figure 7.22 Frequency Distribution of Discontinuity Persistence in Section A

Figure 7.23 Probability Density Histogram of Discontinuity Persistence in Section B

Figure 7.24 Probability Density Histogram of Discontinuity Length in Section A

Figure 7.25 Frequency Distribution of Discontinuity Length in Section B
Table 7.6 Results of frequency distribution of discontinuity lengths in sections A and B

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of Values</th>
<th>Total length of scan-line L (m)</th>
<th>Mean (m)</th>
<th>Std. Div</th>
<th>&lt;0.5 m (%</th>
<th>Over 3 m (%</th>
<th>Maximum concentration at</th>
<th>Mode (%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>450</td>
<td>79.32</td>
<td>1.67</td>
<td>1.01</td>
<td>6.44</td>
<td>28</td>
<td>3 - 3.5</td>
<td>28</td>
</tr>
<tr>
<td>Section B</td>
<td>422</td>
<td>76.27</td>
<td>1.79</td>
<td>1.01</td>
<td>5.92</td>
<td>31.75</td>
<td>3 - 3.5</td>
<td>31.75</td>
</tr>
<tr>
<td>Total</td>
<td>872</td>
<td>155.59</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

7.9.10 Discontinuity Spacing

The frequency distribution of discontinuity spacing in sections A and B was examined for a basalt rock mass at the Dunmore Quarry. For each section of the quarry, three dimensional geological mapping was carried out in order to consider the characteristics of discontinuities and their effects on the stability of slope faces. In addition, the data was used in statistical analysis in order to establish the relationships between discontinuity characteristics.

From a total of 15 separate scan-line surveys with a total line length of 155.59 meters, some 872 discontinuities were read (vertically and horizontally) from the rock exposures on the slope faces. The data obtained from each site were plotted in the form of a histogram using a spacing class interval of 0.1 metre. Figures 7.26 and 7.27 present the probability density histograms of discontinuity spacings for sections A and B. For each data set, the least squares best fit curve was computed for a negative exponential function of the form

\[ f(x) = ae^{-bx} \] (7.2)
Figure 7.26 Probability Density Histogram of Discontinuity Spacing in Section A

Figure 7.27 Frequency Distribution of Discontinuity Spacing in Section B
Figure 7.28 Probability Density Histogram of 872 Discontinuity Spacings of a Basalt Rock Mass in Dunmore Quarry
for an $a > 0$, and is superimposed on the histogram. Figure 7.28 presents the discontinuity spacing histogram of data derived from 872 discontinuities with a superimposed best fit curve. The correlation coefficient for the best fit curve is ($r^2 = 0.98$). A summary of the field data and best fit curve constant parameters for sections A and B are presented in Table 7.7.

Table 7.7 Summary of field data and negative exponential curve parameters in basalt quarry

<table>
<thead>
<tr>
<th>Sources of data</th>
<th>Total scanline length (L (m))</th>
<th>Number of discontinuities</th>
<th>Theoretical curve parameter ($\lambda = n/L$)</th>
<th>Best fit curve parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>79.32</td>
<td>450</td>
<td>5.67</td>
<td>47.706</td>
</tr>
<tr>
<td>Section B</td>
<td>76.27</td>
<td>422</td>
<td>5.53</td>
<td>64.841</td>
</tr>
<tr>
<td>Total data</td>
<td>155.59</td>
<td>872</td>
<td>5.60</td>
<td>73.81</td>
</tr>
</tbody>
</table>

The results from the statistical analysis of discontinuity data showed that, in section A, more than 62.5% of discontinuities have a spacing value of less than 0.2 metre. Results also indicated that, more than 63% of discontinuities have a spacing value of less than 0.2 metre. It should be noted that statistical analysis of discontinuity spacings in this quarry showed that more than 91% of discontinuities in section A and around 90% of discontinuities in section B have a spacing value of less than 0.4 metre.

The mean value of discontinuity spacings was found to be 0.176 m for section A and 0.18 m for section B. The values of mean, and the percentages of discontinuity spacings for under 0.4 metre, show that the rock mass is heavily jointed as presented in Figure 7.29. Details of the results of the statistical analysis of discontinuity spacings for sections A and B of Dunmore Quarry are presented in Table 7.8.
Figure 7.29 General view of a heavily jointed basalt rock mass in Dunmore Quarry.
Table 7.8  Result of statistical analysis of discontinuity spacings in Dunmore Quarry.

<table>
<thead>
<tr>
<th>Structural regions</th>
<th>Number of Values</th>
<th>Mean</th>
<th>Std. Div</th>
<th>&lt;0.2 metre (%)</th>
<th>Over one metre (%)</th>
<th>Maximum concentration at</th>
<th>Mode (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>450</td>
<td>0.17</td>
<td>0.17</td>
<td>62.89</td>
<td>0.44</td>
<td>0.0 - 0.2</td>
<td>62.89</td>
</tr>
<tr>
<td>Section B</td>
<td>422</td>
<td>0.18</td>
<td>0.17</td>
<td>63.03</td>
<td>0.24</td>
<td>0.0 - 0.2</td>
<td>63.03</td>
</tr>
</tbody>
</table>

7.9.11 Rock Quality Designation (RQD) in Dunmore Quarry

In this chapter, the values of actual RQD, calculated from the joint survey (spacing values) and theoretical Rock Quality (RQD*) were calculated, with the results being presented in Table 7.9.

Table 7.9 Comparison between measured and theoretical RQD in a basalt rock mass

<table>
<thead>
<tr>
<th>Source of data</th>
<th>Length (L (m))</th>
<th>Average discontinuity frequency (λ)</th>
<th>Measured RQD %</th>
<th>Theoretical RQD* %</th>
<th>Differences in RQD values (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>79.32</td>
<td>5.67</td>
<td>80.04</td>
<td>89.65</td>
<td>+9.61</td>
</tr>
<tr>
<td>Section B</td>
<td>76.27</td>
<td>5.53</td>
<td>83.06</td>
<td>90.16</td>
<td>+7.10</td>
</tr>
<tr>
<td>Total</td>
<td>155.59</td>
<td>5.60</td>
<td>81.52</td>
<td>89.88</td>
<td>+8.63</td>
</tr>
</tbody>
</table>

From Table 7.9 it can be concluded that, RQD* can be calculated to within 9.6% of the actual RQD obtained from the joint survey data (using the average number of discontinuity per metre (λ)). The terms Rock Quality Percentage (RQP) and Rock Quality Risk (RQR) were proposed by Sen (1990). RQP and RQR are defined as probability quantities and they may assume any value between 0 to 1 inclusively. The RQD provides a reasonable
evaluation of rock mass quality but, for a fuller understanding of the behaviour of a rock mass, it is necessary to examine the rock masses using the terms RQP and RQR. It was decided to use these terms for evaluation of the basalt rock mass in Dumore Quarry.

The quality chart (Figure 5.24) proposed by Sen (1990) was used to evaluate the quality of the rock mass. This chart provides the essential relationships between quantitative parameters such as (RQD), (RQP), (RQR), and qualitative regions for an effective description of the rock mass. The results are presented in Table 7.10.

Table 7.10 Different Values of (RQD), (RQP), and (RQR) for a basalt rock mass.

<table>
<thead>
<tr>
<th>Source of data</th>
<th>Average discontinuity frequency (λ)</th>
<th>RQD (%)</th>
<th>RQP</th>
<th>RQR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section A</td>
<td>5.67</td>
<td>80.04</td>
<td>Good - Fair 0.05 - 0.95</td>
<td>0.95</td>
</tr>
<tr>
<td>Section B</td>
<td>5.53</td>
<td>83.06</td>
<td>Good - Fair 0.08 - 0.92</td>
<td>0.92</td>
</tr>
<tr>
<td>Total</td>
<td>5.60</td>
<td>81.52</td>
<td>Good - Fair 0.06 - 0.94</td>
<td>0.94</td>
</tr>
</tbody>
</table>

It is clear from Table 7.10 that section A, with a RQD of 80.04% and λ = 5.67, is a rock mass with a double description mode of “good” and “fair” quality, respectively. It should be noted that this description applies to this particular λ = 5.67 and RQD = 80.04. In other word, with the same RQD (80%) and λ = 6, for a quantitative description of these qualities, one can find RQP values from the chart of “good” and “fair” descriptions of 0.02 and 0.98 respectively. In this case, the greatest risk is (0.98) which is not representative of the rock mass.
CHAPTER SEVEN Application of Modified Rock Mass Classification on Stability of Basalt Quarry

7.10 ANALYTICAL STATISTICS

7.10.1 Factor Analysis and Data Acquisition

An effort was made to ascertain the degree of importance of each type of discontinuity parameter, in relationship with their effect on the slope stability of a basalt rock mass. Factor analyses were carried out on 872 discontinuities, gathered from different parts of the Dunmore Quarry. The stability of hard rock slopes is dependent on discontinuity parameters (as independent variables). In order to evaluate the effect of these independent variables on the stability of slope faces of open pit mines, some 872 discontinuities in basalt rock mass were examined using the factor analysis method. The contributing variables were categorised according to the relationship between each variable and the slope stability phenomena.

When the discontinuity parameters were analysed, four factors yielded an eigenvalue greater than one, accounting for 62.90% of variance. The first factor explains 28.6% of the variance, the second 13.7%, the third 13.2%, and the fourth 11.4%. A correlation matrix was calculated between variables to determine the strength of the relationship between each variable. The results are presented in Table 7.11.

Table 7.11 Correlation matrix of discontinuities parameters in the slope stability analysis of Dunmore Quarry

<table>
<thead>
<tr>
<th>Variables</th>
<th>X1</th>
<th>X2</th>
<th>X3</th>
<th>X4</th>
<th>X5</th>
<th>X6</th>
<th>X7</th>
<th>X8</th>
</tr>
</thead>
<tbody>
<tr>
<td>(X1) = Dip angle</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(X2) = Aperture</td>
<td>0.12*</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(X3) = Curvature</td>
<td>0.09*</td>
<td>0.28*</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(X4) = Roughness</td>
<td>0.22*</td>
<td>0.20*</td>
<td>0.22*</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(X5) = Persistence</td>
<td>0.12</td>
<td>0.09*</td>
<td>0.09*</td>
<td>0.12*</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(X6) = Infill - Materials</td>
<td>0.16*</td>
<td>0.11*</td>
<td>0.17*</td>
<td>0.18*</td>
<td>0.15*</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(X7) = Spacing</td>
<td>0.08*</td>
<td>0.01</td>
<td>0.02</td>
<td>0.12*</td>
<td>0.09*</td>
<td>0.06*</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>(X8) = Length</td>
<td>0.27*</td>
<td>0.20*</td>
<td>0.21*</td>
<td>0.74*</td>
<td>0.13*</td>
<td>0.18*</td>
<td>0.09*</td>
<td>1</td>
</tr>
</tbody>
</table>

*P<0.05 Significant at 95%
As is clear from Table 7.11 there is a significant relationship between most of the variables (p<0.05). In the other word it was established by the statistical analysis that, these discontinuity parameters are real parameters affecting the stability of hard rock slopes. The results of statistical analysis (Factor analysis) are presented in Table 7.12. This table shows that variable length relates highly to Factor 1 with 28% of the variance. This means that this factor makes a significant contribution to the slope stability of hard rock slopes.

From Table 7.12 it can be seen that other discontinuity parameters such as curvature, persistence, and spacing make a significant contribution to the slope stability of hard rocks. From the results of statistical analysis practically it was concluded that the length of discontinuity has a significant effect on the stability of hard rock slopes.

Table 7.12  Factor loading of the stability evaluation from the contributor factors

<table>
<thead>
<tr>
<th>Independent variables</th>
<th>Factor 1</th>
<th>Factor 2</th>
<th>Factor 3</th>
<th>Factor 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dip angle</td>
<td>0.27</td>
<td>0.24</td>
<td>0.41</td>
<td>0.20</td>
</tr>
<tr>
<td>Aperture</td>
<td>0.06</td>
<td>0.62</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>Curvature</td>
<td>0.09</td>
<td>0.64</td>
<td>0.03</td>
<td>0.06</td>
</tr>
<tr>
<td>Roughness</td>
<td>0.52</td>
<td>0.01</td>
<td>0.13</td>
<td>0.04</td>
</tr>
<tr>
<td>Persistence</td>
<td>0.17</td>
<td>0.01</td>
<td>0.60</td>
<td>0.16</td>
</tr>
<tr>
<td>Infilling materials</td>
<td>0.08</td>
<td>0.05</td>
<td>0.57</td>
<td>0.10</td>
</tr>
<tr>
<td>Spacing</td>
<td>0.14</td>
<td>0.01</td>
<td>0.04</td>
<td>0.97</td>
</tr>
<tr>
<td>Length</td>
<td>0.53</td>
<td>0.04</td>
<td>0.08</td>
<td>0.02</td>
</tr>
<tr>
<td>Variance</td>
<td>0.28</td>
<td>0.14</td>
<td>0.13</td>
<td>0.11</td>
</tr>
<tr>
<td>Eigenvalue</td>
<td>2.29</td>
<td>1.1</td>
<td>1.06</td>
<td>1.01</td>
</tr>
</tbody>
</table>
From the results of statistical analysis it was found that, amongst the contributing factors (dip angle, aperture, infilling materials, curvature, water condition, spacing, persistence, and length of discontinuities) affecting rock slope stability of hard rocks, the length of discontinuities is the most important parameter affecting hard rock slope stability. The curvature of discontinuities, persistence and spacing are other significant factors contributing to the rock mass behaviour.

7.11 STABILITY ASSESSMENT AND GRAPHICAL PRESENTATION OF DISCONTINUITIES

7.11.1 Stability Assessment of the Slope Face in Section A

For the stability assessment of rock slope face in this section, some 450 discontinuities were measured using the three dimensional joint survey (scan-line) method. Discontinuities were measured in vertical and horizontal directions. For the measurement of discontinuities in vertical direction, a cherry picker was used, as is presented in Figure 7.30.

For the stability assessment of the slope face in this section of the quarry, a lower hemisphere equal area projection method was used. A pole plot and the contour plots of discontinuities of section A are presented in Figure 7.31.

According to the high density pole projection of 450 discontinuities, three major discontinuity planes were identified in this section. The orientation of these joints sets (dips and dip directions) are 83°/024, 81°/092, and 83°/297 respectively. A polar net superimposed on the contoured pole plot net as well as major discontinuity planes are presented in Figure 7.32. A friction angle of 26.24 degrees with a slope face of 80 degrees and dip direction of 100 degrees were used in order to assess the stability of the slope face.
Figure 7.31 Analysis of discontinuity orientation data of section A from Dunmore Quarry
(a) Pole plot net
(b) Contoured plot net.
Figure 7.32 Analysis of pole plot data of section A from Dunmore Quarry. (a) A polar net superimposed on the contoured pole plot net. (b) Stereographic projection of Major discontinuity planes, slope face and friction angle.
Stability analysis for this section showed that, instability occur due to the presence of three major joint sets in this part of the quarry. As is clear from Figure 7.32 the intersecting lines I\textsubscript{23} is situated within the shaded area. It can lead to wedge failure along the intersecting lines in section A of the quarry. It should be noted that there is a potential of sliding due to the presence of joint sets 2 and 3 in this section of the quarry. As part of remedial work and in order to prevent potential failure in this section, changing the slope orientation to 60/130 is recommended for reducing the risk of wedge failure along the intersecting lines I\textsubscript{23}.

7.11.2 Stability Assessment of the Slope Face in Section B

A stability assessment has been carried out for this particular section of the quarry and based on the high density pole concentration of 422 discontinuities (Figure 7.33) measured from the slope face in section B of the quarry, five dominant discontinuity planes were identified. The orientation of these joint sets (dips and dip directions) are 82.225, 84.207, 83.303, 35.208, and 07.203 respectively. For stability analysis of the slope face of the basalt rock mass, a slope face of 80 degrees with a dip direction of 190 degrees and a friction angle of 26.24 degrees derived from the direct shear testing of the naturally jointed samples were used.

As is clear from Figure 7.34(b), instability of the slope face in this section may occur due to the presence of four joint sets (joint sets 1, 2, 3, 4). The intersecting lines I\textsubscript{12} is situated within the shaded area and it can lead to wedge failure along the intersecting lines in this part of the quarry.

It should be noted that joint sets 3 and 4 do not affect the stability of the slope face in this section but there is a potential of sliding due to the presence of these joint sets in this part of the quarry.
Figure 7.33 Illustration of discontinuity orientation data of section B from Dunmore Quarry. (a) Pole plot net. (b) Contoured plot net
Figure 7.34 Analysis of cluster pole of section B from Dunmore Quarry. (a) A polar net superimposed on the contoured pole plot net. (b) Stereographic projection of dominant discontinuity planes, slope face and friction angle.
In order to prevent potential failure in this part of the quarry (and as part of remedial work) changing the slope face orientation to 70/170 can prevent the potential of wedge failure along the intersecting lines I$_{12}$. The orientations of dominant discontinuity joint sets are presented in Table 7.13.

Table 7.13 Orientations of major Discontinuity planes and Slope Faces

<table>
<thead>
<tr>
<th>Structural region (Dip - dip direction)</th>
<th>Section A</th>
<th>Section B</th>
</tr>
</thead>
<tbody>
<tr>
<td>First predominant joint set</td>
<td>83/024</td>
<td>82/225</td>
</tr>
<tr>
<td>Second predominant joint set</td>
<td>81/092</td>
<td>84/207</td>
</tr>
<tr>
<td>Third predominant joint set</td>
<td>83/297</td>
<td>83/303</td>
</tr>
<tr>
<td>Fourth predominant joint set</td>
<td>--------</td>
<td>35/208</td>
</tr>
<tr>
<td>Fifth predominant joint set</td>
<td>--------</td>
<td>07/203</td>
</tr>
<tr>
<td>Slope face</td>
<td>80/100</td>
<td>80/190</td>
</tr>
<tr>
<td>Changed slope face</td>
<td>60/130</td>
<td>70/170</td>
</tr>
</tbody>
</table>

7.12 ROCK MASS CHARACTERISATION AT DUNMORE QUARRY

In order to characterize rock mass at Dunmore Quarry the following steps have been carried out:

- Determining engineering properties of intact rock in laboratory
• Estimation of rock mass strength from laboratory and field data

• Classification of rock mass using
  o RMR system
  o Q system, and
  o Weakening Coefficient system

7.12.1 Engineering Properties of Intact Rock

The results of laboratory tests are presented in Table 7.14 which were used in rock mass classification as input data. Moreover, direct shear tests were carried out on naturally jointed samples in order to achieve the basic internal friction angle and coefficient cohesion as input data for the stability assessment of slope faces.

Table 7.14 Engineering properties of intact rock samples (basalt) from Dunmore Quarry

<table>
<thead>
<tr>
<th>Engineering properties of intact rock</th>
<th>Structural regions A and B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial compressive strength (MPa)</td>
<td>182.11</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>9.42</td>
</tr>
<tr>
<td>UCS derived from diametral point load test (MPa)</td>
<td>212.80</td>
</tr>
<tr>
<td>UCS derived from axial point load test (MPa)</td>
<td>208.64</td>
</tr>
<tr>
<td>Poisson's ratio (MPa)</td>
<td>0.24</td>
</tr>
<tr>
<td>Elastic modulus (GPa)</td>
<td>39.3</td>
</tr>
<tr>
<td>Friction angle</td>
<td>57.86</td>
</tr>
<tr>
<td>Bulk density kg/m$^3$</td>
<td>2710.17</td>
</tr>
</tbody>
</table>
7.12.2 Estimation of Rock Mass Strength Parameters

In order to evaluate the rock mass strength parameters, some natural joint samples from the exposures on the slope faces were collected and tested using a direct shear box. From the results of direct shear tests carried out on naturally jointed samples, the basic internal friction angle was found to be 26.24 degrees for sections A and B of the quarry and the cohesion strength was found to be 2.02 MPa for these samples. It should be noted that, the internal friction angle was used in the stability assessment of the slope faces of the basalt quarry, as explained in previous sections.

7.12.3 Rock Mass Classification of Dunmore Quarry

7.12.3.1 Results of geomechanics classification system (RMR)

The basalt rock mass in Dunmore quarry was divided into two sections. For each section, the first five classification parameters were determined based on the laboratory tests and joint survey programs. The rating of each individual factor were quantified according to the suggested ranges of values in section (a) of Table 5.15 proposed by Bieniawski (1973). As is clear from Equation 7.4 the orientation of discontinuities has significant effects on the behaviour of the rock masses and its rating should be subtracted from the sum of the first five parameters.

The results of the RMR classification for the basalt rock mass is presented in Table 7.15. Based on the results of RMR system, the quality of basalt rock mass was identified as a poor quality rock mass in section A and a fair quality rock mass for section B of the quarry.
Table 7.15 Geomechanics Classification (RMR) of Porphyry Rock Mass in Dunmore Quarry

<table>
<thead>
<tr>
<th>Rock Mass Parameters</th>
<th>Section A</th>
<th></th>
<th>Section B</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>U. C. S. (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Value</td>
<td>182.11</td>
<td>12</td>
<td>182.11</td>
<td>12</td>
</tr>
<tr>
<td>RQD %</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Value</td>
<td>80.05</td>
<td>17</td>
<td>83.06</td>
<td>17</td>
</tr>
<tr>
<td>Discontinuity Spacing (m)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Value</td>
<td>0.176</td>
<td>8</td>
<td>0.18</td>
<td>8</td>
</tr>
<tr>
<td>Discontinuity Condition</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class IV</td>
<td>10</td>
<td></td>
<td>Class IV</td>
<td>10</td>
</tr>
<tr>
<td>Ground Water Condition</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dry</td>
<td>15</td>
<td></td>
<td>Dry</td>
<td>15</td>
</tr>
<tr>
<td>Total rating</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Value</td>
<td>62</td>
<td></td>
<td>62</td>
<td></td>
</tr>
<tr>
<td>Orientation Rating</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class III</td>
<td>-25</td>
<td></td>
<td>Class II</td>
<td>-5</td>
</tr>
<tr>
<td>Total RMR Rating</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Value</td>
<td>37</td>
<td></td>
<td>57</td>
<td></td>
</tr>
</tbody>
</table>

7.12.3.2 Results of Rock Mass Quality (Q System) in Dunmore Quarry

Rock mass quality (Q system) were used in order to evaluate the quality of the basalt rock mass in Dunmore Quarry. The results of Q index are presented in Table 7.17. Based on the rating obtained from different parameters, the quality of the rock mass was found to be poor in section A and fair in section B of the quarry.
Table 7.16 New Rating for Geomechanics Classification of Jointed Rock Masses

A: Classification Parameters and Their Ratings

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Ranges of Values</th>
<th>Rating</th>
<th>For this low range uni-axial compressive test is preferred</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength of intact rock material</td>
<td>&gt;10 Mpa</td>
<td>25</td>
<td>5-35 Mpa</td>
</tr>
<tr>
<td>Point Load strenght index</td>
<td>4-10 Mpa</td>
<td>20</td>
<td>1 - 5 Mpa</td>
</tr>
<tr>
<td>Uniaxial compressive strength</td>
<td>2-4 Mpa</td>
<td>15</td>
<td>&lt;1 Mpa</td>
</tr>
<tr>
<td>&gt;250 Mpa</td>
<td>1-2 Mpa</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>&gt;150 Mpa</td>
<td>0-1 Mpa</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Drill Core quality (RQD)</td>
<td>90%-100%</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Uniaxial compressive strength</td>
<td>75%-90%</td>
<td>17</td>
<td></td>
</tr>
<tr>
<td>For this low range uni-axial compressive test is preferred</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uniaxial compressive strength</td>
<td>50%-75%</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>&gt;25% to 50%</td>
<td>25%-50%</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>&lt;25%</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spacing of discontinuities</td>
<td>&gt;2 m</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>0.6-2 m</td>
<td>12</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>200-600 mm</td>
<td>8</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>60-200 mm</td>
<td>&lt;60 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition of discontinuities</td>
<td>Very rough surfaces</td>
<td>25</td>
<td>Complete dry</td>
</tr>
<tr>
<td>Not continuous</td>
<td>Slightly rough surfaces</td>
<td>20</td>
<td>Damp</td>
</tr>
<tr>
<td>Separate &lt; 1 mm</td>
<td>Slightly weathered walls</td>
<td>15</td>
<td>Wet</td>
</tr>
<tr>
<td>Unweathered wall rock</td>
<td>Highly weathered walls</td>
<td>10</td>
<td>Dripping</td>
</tr>
<tr>
<td>Slickensided surfaces or Gouge &lt;5 mm thick or Separation &gt;5 mm continuous</td>
<td>5</td>
<td>Flowing</td>
<td></td>
</tr>
<tr>
<td>Soft gouge &gt;5 mm thick or Separation &gt;5 mm continuous</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ground Water</td>
<td>Inflow per 10 m tunnel length</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>None or &lt;10 L/min</td>
<td>10-25 L/min</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>or 25-125 L/min</td>
<td>or &gt;125 litres/min</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Joint water Pressure</td>
<td>0</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>Ratio = Major principal stress</td>
<td>0.0 - 0.1</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>General condition</td>
<td>0.1 - 0.2</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>Completely dry</td>
<td>0.2 - 0.5</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Damp</td>
<td>&gt; 0.5</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Wet</td>
<td>Continuous</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dripping</td>
<td>Flowing</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 7.17 Engineering Rock Mass Quality (Q System) of the Dunmore Quarry.

<table>
<thead>
<tr>
<th>Rock Mass Parameters</th>
<th>Structural region (A)</th>
<th>Structural region (B)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value</td>
<td>Rating</td>
</tr>
<tr>
<td>Rock Quality Designation (RQD) %</td>
<td>80.05</td>
<td>83.3</td>
</tr>
<tr>
<td>Joint Set Number (Jn)</td>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>Shear Strength Factor (Jr/Jm)</td>
<td>1.5/2</td>
<td>0.75</td>
</tr>
<tr>
<td>Joint Water Reduction Factor (Jw)</td>
<td>Dry</td>
<td>1</td>
</tr>
<tr>
<td>Rating Adjustment Factor (RAF)</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Total Rating</td>
<td>1.33</td>
<td></td>
</tr>
<tr>
<td>Description</td>
<td>Poor</td>
<td></td>
</tr>
</tbody>
</table>

#### 7.12.3.3 Results of Weakening Coefficient Classification System in Dunmore Quarry

The parameters contributing in Weakening Coefficient rock mass classification and their rating as well as the rating adjustment regarding discontinuity orientations for Dunmore quarry are presented in Table 7.18. According to the results obtained from the Weakening Coefficient rock mass classification, the quality of the basalt rock mass was found to be poor quality in section A and moderate quality for section B of the quarry.
It should be noted that, based on field observations (comparison of different rock samples in different parts of the quarries from point of view of weathering and alteration degrees) and laboratory testing results (achieving different value of uniaxial compressive strength for different samples with different degrees of weathering) it was concluded that the ratings and categories given to the uniaxial compressive strength of intact rock, rating for spacing of discontinuities, and rating for condition of discontinuities in RMR system are not consistent with the realistic behaviour and condition of rock samples. Therefore a new rating for the RMR system of rock mass classification was proposed as is presented in Table 7.18

Table 7.18 The Results of Weakening Coefficient Classification of Limestone in Dunmore Quarry

<table>
<thead>
<tr>
<th>Rock Mass Parameters</th>
<th>Section A</th>
<th>Section B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Value</td>
<td>Rating</td>
</tr>
<tr>
<td>RQD %</td>
<td>80.03</td>
<td>0.80</td>
</tr>
<tr>
<td>Discontinuity Spacing (m)</td>
<td>0.176</td>
<td>0.7</td>
</tr>
<tr>
<td>Joint Surface</td>
<td>Rough</td>
<td>0.9</td>
</tr>
<tr>
<td>Joint Infilling</td>
<td>Open &lt;5 mm</td>
<td>0.7</td>
</tr>
<tr>
<td>Discontinuity Aperture</td>
<td>2 - 6 mm</td>
<td>0.7</td>
</tr>
<tr>
<td>Weakening Coefficient (WC)</td>
<td>0.247</td>
<td>0.366</td>
</tr>
<tr>
<td>Orientation Rating</td>
<td>0.247 x 0.37</td>
<td>0.366 x 0.37</td>
</tr>
<tr>
<td>Total Weakening Coefficient (WC)</td>
<td>0.092</td>
<td>0.135</td>
</tr>
<tr>
<td>Description</td>
<td>Poor</td>
<td>Moderate</td>
</tr>
</tbody>
</table>
7.13 SUMMARY OF ENGINEERING EVALUATION OF THE BASALT ROCK MASS IN DUNMORE QUARRY

Based on the laboratory testing results, field investigations and results of different rock mass classifications, most important engineering properties of the basalt rock mass in Dunmore quarry are summarised in Table 7.19.

Table 7.19 Engineering Description of basalt rock mass in Dunmore Quarry

<table>
<thead>
<tr>
<th>Property</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>Fine grained, extrusive, igneous rock (latite) similar to basalt with higher silica</td>
</tr>
<tr>
<td>Rock mass</td>
<td>Jointed rock mass, containing typical columnar structures, zeolite filled joint planes</td>
</tr>
<tr>
<td>Rock Quality Designation (RQD)</td>
<td>80.04 in section A and 83.06 in section B</td>
</tr>
<tr>
<td>Joint set number and orientations</td>
<td>Three dominant joint sets in section A and Five major joint sets in section B</td>
</tr>
<tr>
<td>Joint roughness</td>
<td>Mainly rough in sections A and B</td>
</tr>
<tr>
<td>Water condition</td>
<td>Dry rock mass</td>
</tr>
<tr>
<td>Rock mass rating and quality</td>
<td>37 - 57 (poor rock mass in section A and fair rock quality in section B)</td>
</tr>
<tr>
<td>Cohesion and Friction angle of the rock mass (Shear strength parameters)</td>
<td>Cohesion of 100 - 200 kPa and friction angle of 15- 25° for section A and cohesion of 200 - 300 kPa and friction angle of 25 - 30° in section B</td>
</tr>
</tbody>
</table>
7.14 CONCLUSIONS

Basalt rock mass at Dunmore Quarry was examined in order to assess the stability of slope faces in different parts of the quarry. Based on the field observations and geological mapping, Dunmore Quarry consists of a good quality rock material as is used for the production of rail ballast, aggregate and material of low shrinkage concrete.

The frequency distribution of discontinuity parameters (dip angle, orientation, curvature, infilling materials, roughness, termination, length, and discontinuity spacing) were obtained using different statistical techniques in this quarry. The results have shown that more than 75% of the discontinuities have a dip angle of over 60 degrees and the mean value of spacing was found to be between 2 - 6 mm. It was also found that the mean value of the discontinuity length in section A is 1.67m and 1.79 m for section B. The mean value of discontinuity spacing was found to be 0.18 m for sections A and B. In addition, factor analysis showed that the length of discontinuity is the most important factor affecting rock slope stability in this quarry.

Results of the stability assessment in Dunmore Quarry showed that three dominant joint sets in section A and five major joint sets in section B of the quarry. It was also identified that there is a potential of wedge failure in sections A and B of the quarry.

Results of different rock mass classifications carried out in Dunmore Quarry showed a different quality of rock mass from fair to poor in different sections. The Values of the cohesion and friction angle for the basalt rock mass were found to be 100 - 200kPa and 15 to 25° for section A and 200 - 300 kPa and 25- 35° in section B respectively.
Chapter 8
Conclusions and Recommendations
CHAPTER EIGHT

CONCLUSION AND RECOMMENDATIONS

8.1 SUMMARY

The stability of hard rock slopes in open pit mines has been comprehensively investigated during the course of this research. The thesis has presented a methodology for, and results of, laboratory testing, field investigations and computer analyses which were carried out on three different open pit mines.

The purpose of this thesis was to study the effect of rock mass characteristics on slope stability in surface mining; with special reference to quarrying in NSW, Australia. To achieve these aims, extensive laboratory tests and field investigations were carried out on three different type of rocks (porphyry, limestone, and basalt) in three NSW quarries.

In order to evaluate the potential modes of failure within the rock masses, sufficient discontinuity data is required. For the purpose of this research, necessary data were obtained from a three dimensional scan-line joint survey carried out on the slope faces of rock exposures. A computer analysis of discontinuity orientation data was carried out for the stability assessment of slope faces, using data obtained from geological mapping.
A lower hemisphere stereographic projection method was used for the graphical presentation of discontinuity orientations.

The main aims followed throughout the thesis were:

- Consideration of most important discontinuity parameters affecting hard rock slope stability.
- To study the physical and mechanical properties of rocks in relation with the slope stability.
- To identify major discontinuity parameter governing the failure of hard rock slope stability in open pit mining.
- To assess the behaviour and response of heavily jointed rock masses in surface mining operations.
- To achieve sufficient data from discontinuity parameters for use in designing a safer and economic open pit mine.
- Modification and application of rock mass classification systems for evaluation of rock mass quality.
- Statistical evaluation of physical and mechanical properties of discontinuities and consideration of more significant factor affecting rock slope stability of open pit mines.

8.2 CONCLUSION

Theoretical, empirical and analytical techniques relating to the slope stability of hard rock slopes were reviewed, and the following conclusions were obtained:

- Statistical analysis of the discontinuity data provides sufficient information relating to the distribution of discontinuity parameters in open pit mines.
- The three dimensional scan-line method is efficient for joint surveying and data collection from rock mass discontinuities.
• Results of the studies from different types of hard rock masses in all sites showed that the stability of hard rock slopes is significantly governed by the presence of geological structures.

• The shear strength of the discontinuity planes within the rock mass have a significant effect on the stability of hard rock slopes.

• The behaviour of rock masses are governed by the slope geometry (i.e. discontinuity orientation, discontinuity spacing, dip angle and the orientation of slope face or the orientation of excavation).

It has been shown by laboratory and field studies carried out at three open pit mines that the following major factors influence stability of hard rock slopes:

• Discontinuity orientations
• Dip of the discontinuity
• Joint spacing
• Shear strength of discontinuity planes
• Length of discontinuity
• Rock Quality Designation (RQD)
• Rating adjustment parameter related to discontinuity orientation
• Discontinuity roughness
• Discontinuity infilling materials

8.3 APPLICATION AND MODIFICATION OF ROCK MASS CLASSIFICATION SYSTEMS

This study led to the use of quantitative and qualitative descriptions of hard rock masses. For the purpose of stability assessment, and in order to examine the quality of different types of rock masses, stereographic projection method and three current rock mass
classifications were used. From the stability analysis of three surface mines it has been concluded that there are several modes of failure for hard rock slope faces in open pit mining. They are as follows;

- No potential of failure (stable)
- The potential of failure
- The potential of one mode of failure
- The potential of two modes of failure
- The potential of several modes of failure

Attention was drawn to the application of rock mass classification systems in order to quantitatively assess the quality of different rock masses. The CSIR classification system (RMR index) and NGI system (Q index) as well as the Weakening Coefficient classification (WC) system were used for evaluation of rock mass quality.

This thesis has shown that the results obtained for a particular rock mass, from different rock mass classifications, are very close to each other. Based on field observations and laboratory testing results, it has been concluded that the ratings given by Bieniawski, (1974) to the uniaxial compressive strength of intact rock (zero to fifteen), discontinuity spacing, and discontinuity condition rating are not very consistent with the realistic behaviour of rock and condition of rock samples. For example, in the case of uniaxial compressive strength and in the range of 100 to 250 MPa some intact rock samples with different degrees of weathering exhibit different values of uniaxial compressive strength with a reasonable variation. In addition, it seems that the rating given to uniaxial compressive strength is lowly rated in comparison with other geotechnical parameters affecting the behaviour of rock masses.

The rating proposed for uniaxial compressive strength of intact rock includes a long range of uniaxial compressive strength for intact rocks which show very visible differences. The rock samples which are moderately weathered have the same rating as
the samples which are fresh, with a high value of uniaxial compressive strength. Therefore in thesis the RMR rating was modified and a new rating (Table 7.16) for the RMR classification system was proposed by the author.

8.4 FIELD INVESTIGATION

This research has once again shown that results obtained from the field investigations provide valuable information of geological structural features and rock mass discontinuities. For the purpose of this thesis, extensive field investigations have been carried out in three different open pit mines. The main aim of field investigations was preparation of the required data from the rock masses used in the stability assessment of hard rock slopes and rock mass classification of the rock masses. In each site, a three dimensional scan-line joint survey has been carried out on the slope faces of rock exposures. It has been identified that the three dimensional joint survey is a suitable and cost effective method for collecting data from the discontinuities within the rock masses.

8.5 LABORATORY TESTING OF DIFFERENT TYPE OF ROCKS

The important rock properties used for mine design are the modulus of elasticity and the compressive, tensile and shear strength which are often measured in laboratory. Extensive laboratory tests were undertaken to investigate the effect of physical and mechanical properties of rocks on the stability of hard rock slope faces in open pit mines. Mechanical properties of the rocks, including complete stress-strain curves were obtained by mean of uniaxial and triaxial compressive strength, direct shear, point load and Brazilian tests. In this thesis, factors affecting the shear strength of discontinuity surfaces and an interpretation of the shear strength of naturally jointed samples of different type of rocks, together with discontinuity data collected from the rock masses, have been reviewed.

Conventional failure criteria (Mohr Coulomb and Power Law) were examined against the direct shear test results derived from the testing of naturally jointed samples. The results
showed that both linear and power law criteria are close and parallel to each other and both of them have shown very high correlation coefficients with the experimental results. It also was found that, these failure criteria are a simple and close model for measuring the shear strength of naturally jointed samples in a laboratory. Based on the direct shear test results obtained from naturally jointed samples, it was concluded that in the lack of in-situ direct shear tests, the results of laboratory testing of naturally jointed samples can be used for evaluating the stability of hard rock slopes, if the samples can be representative of the rock mass.

The relationships between various laboratory and index tests were examined using different statistical techniques. Some of the regression methods (such as linear, power, and exponential regression) were examined in order to find the relationships between laboratory and index tests for hard rocks. The results obtained from these correlations showed that there is a very good relationship between the laboratory and index tests for different types of hard rock.

8.6 STATISTICAL ANALYSIS OF LABORATORY AND FIELD DATA

Statistical analysis methods were used to consider the importance of, discontinuity parameters and their interactions, particularly in relation to the stability assessment of open pit mining. Statistical analysis showed that discontinuity data can provide a wide range of information from site investigation and geological mapping of the slope faces of open pit mines.

The statistical analysis of discontinuity parameters (contributing to the stability of hard rock slopes) showed that the instability and modes of failure in hard rock slopes are mainly controlled by discontinuity length. In addition a significant relationship was established between the discontinuity parameters affecting stability of hard rock slopes.
8.7 STABILITY ASSESSMENT OF ROCK SLOPE FACES

This thesis also involved a study of the behaviour of rock masses and rock slope faces using a stability analysis computer program. For the purpose of stability assessment input data were obtained from laboratory testing (direct shear testing on naturally jointed samples) and joint surveying using scan-line method. It has been shown that computer analysis methods can be used as a reliable tool for the quantitative assessment of discontinuity orientations for the purpose of slope stability analysis. A computer analysis of discontinuity orientations and their effect on the stability of hard rock slopes was carried out in order to identify potential modes of failure and the orientation of the existing dominant discontinuity planes within the rock mass.

This thesis also indicated the necessity for using accurate and representative data from the mechanical properties of different rock samples in the stability analysis of hard rock slope faces. The result of stability analysis showed that the stereographic projection method is a simple and cost effective method for the stability evaluation of hard rock slopes.

Based on laboratory and field investigations carried out in several open pit mines, it was concluded that there are some major factors affecting slope stability. These are as follows,

- Shear strength parameters
- Discontinuity characteristics (discontinuity dip angle, discontinuity orientation, length, roughness, aperture, and infilling materials)
- Geometrical factors (slope angle, slope height, overall slope and width of the benches)
- Blasting, drilling, and earthquake (dynamics factors)

Laboratory testing, field investigation, statistical analysis techniques, and computer analysis undertaken during the course of this thesis have successfully shown that the stability problems in open pit mines can be quantified and, as a consequence, alternative
and remedial solutions may be used in such a way as to overcome instability problems in open pit mining operations.

8.8 RECOMMENDATIONS FOR FUTURE RESEARCH

The effect of various discontinuity parameters on different aspects of the stability of hard rock slopes in open pit mines was investigated and presented through the thesis. The following areas for future research are recommended based on the achievements of the present research.

The mechanical properties of rocks are important as a realistic input data for the purpose of rock mass classifications. Regarding to the new RMR rating proposed in this thesis, further research is required to practically examined the new rating in different places. Therefore, it is recommended that more investigations should be conducted, particularly in relation to hard rock masses, in order to develop a reliable rock mass classification system for evaluation of hard rock mass quality.

The application of current rock mass classification systems should be extended to take a weathering parameter into account as a significant factor affecting the rock masses, particularly in the case of a reduction of the shear resistance of discontinuity surfaces which act against the sliding or occurrence of different modes of failure within the rock mass.

Rock strength is a significant factor affecting rock mass behaviour particularly in conjunction with the rock slope stability and foundation engineering. RQD and joint spacing both of which are measures of block size. Together these parameters may account 50 points in the RMR system. It is recommended to reduce these points to lower points and increase the points of other parameters such as strength which may have greater effect on the engineering behaviour of rock.

The variations in the behaviour of rock masses from place to place are based on the variation of in-situ properties of rock mass and the variation of existing geological
features within the rock mass. Due to the above points, there are sufficient reasons to suggest why instability problems occur in some same parts of an open pit mine. Consequently, from a practical point of view, it is suggested that more field investigations and in-situ testing of samples as representative of rock masses should be performed to consider the actual and predominant condition, which governed the behaviour of rock masses.

In this research, exponential function modeling has been established for the description of discontinuity spacings in hard rock masses. Further statistical descriptions of rock mass characteristics might lead to more information about the rock mass characteristics used in statistical distribution models regarding discontinuity properties in hard rock masses.

Statistical analysis from discontinuity data has shown that for steeply dipping joints the length of discontinuities has the most significant effect on rock mass behaviour and affects the stability of hard rock slope faces in open pit mines. Further statistical studies in relation to the physical properties of discontinuities are required to find the effect of these properties on the behaviour of rock masses particularly in surface mining operations.

Since the presence of weak bounds such as shale or clay materials within the hard rock masses have a significant effect on the overall stability of the open pit mines it is recommended that more attention should be made to the study of the effect of infilling materials and weak bounds on the stability of open pit mines.

The author believes that these studies can help to provide more information for the simulation of rock masses in order to study some discontinuity properties such as length, spacing, orientation, aperture, infilling materials of the discontinuities which can be modeled for further studies in open pit mining using three dimensional models. More statistical studies are required in order to have a good understanding of the relationship and interactions between different discontinuity parameters acting on the instability of hard rock slopes in open pit mines.
REFERENCES


Au, S. W. C., (1993), "Reversal Shear Box Test for Hong Kong Saprolitic Soils", Quarterly journal of engineering geology V:26 - pp. 233-237


369


376


382


Terzaghi, K., (1946), "Rock Tunneling with Steel Supports", Ohio, Commercial Shearing and Stamping Co.


Dumitru, I., (1990), "Dunmore Quarry Data Bank", Boral Research, Materials Laboratory Report.

391


392


393


Pfleider, E. P., (1968), "Surface Mining", Maple Press, Pennsylvania, USA. 1061 P.


Shu, D. M., and Bhattacharyya, A. K, (1992), "Influence of the Sloping of Ground Surfaces on Mine Subsidence", Proc. of 11th International Conference on Ground Control in Mining, the University of Wollongong, NSW, 7-10 July, pp.475-481.


400


Appendix 1

Statistical definitions
Central tendency

Loosely defined, the "central tendency" of a set of numbers is the tendency of the data to cluster around certain numerical values (quoted by Wagner, 1991). The arithmetic mean ($\bar{X}$) is the sum of the values of all elements in the data set ($\sum X$) divided into the number of total elements in the data set ($n$).

$$\bar{X} = \frac{\sum_{i=1}^{n} x_i}{n} = \frac{x_1 + x_2 + x_3 + \ldots + x_n}{n} \quad (A.1)$$

Median

This characteristic is another measure of the central tendency. This item is the middle most or most central item in the set of numbers and divides the data set into two halves. This number, which is sometimes denoted $\bar{X}$, is the middle value in a set of numbers that has been arranged form the lowest to the highest.

$$M = \text{the} \left( \frac{n+1}{n} \right) \text{th item in the data arrangement.} \quad (A.2)$$

Mode

Is the value which is most often repeated in the data set. It is the number that appears most frequently in the data set and it is presented by the highest point in the distribution curve of a data set.

Dispersion

The measurement of the dispersion of the data in the data set gives some additional information about the reliability of the central tendency obtained for the data set. The problem with measuring the dispersion using range is that it considers the highest and
the lowest values of the distribution and fails to take into account any other observations in the data series.

**Variance**

Since for some pairs of data sets, the range does not differentiate variability, variance is another measure of the dispersion in the data set. It is a measure of the average square distance between the mean and each item in the population and it can be calculated by the following equation;

$$\sigma^2 = \frac{\sum (x - \bar{x})^2}{n} \quad \text{(A.3)}$$

$\sigma^2$ is the variance of the data set  
$x$ is the item or observation  
$\bar{x}$ is the data set mean  
n is the number of the items in the data set. 

The advantage of variance in comparison with dispersion is that it takes all observations into account.

**Standard deviation**

For making a useful measurement of deviation, usually the square root of variance is used as standard deviation. This item determines with a great deal of accuracy, where the values of the frequency distribution are located in relation to the mean. The standard deviation can be calculated from the following equation;

$$s = \sqrt{\frac{\sum (x_i - \bar{x})^2}{n-1}} \quad \text{(A.4)}$$

This formula is fairly easy to work with as long as n is small (not too many data in the set) and the mean, $\bar{x}$, is an integer. But if $\bar{x}$ is not an integer, the calculation of subtracting it from each of the numbers in the data set, which usually are whole numbers, is cumbersome. Wagner, 1991 suggested following formula to use for
calculation of the standard deviation;

\[ S = \sqrt{\frac{\sum x_i^2 - \left(\frac{\sum x_i}{n}\right)^2}{n-1}} } \]  \hspace{1cm} (A.5)

Where;

\[ \sum x_i^2 \] is the sum of the square of each of the data points, and

\[ (\sum x_i)^2 \] is the square of the sum of the data points.

The standard deviation, taken along with the mean, can be used to provide a measure of variability within a single data set, as well as a contrast between two data sets.

**Standard error**

To provide a statement of error in the estimations, the standard error of the mean is defined as follows:

\[ \sigma_x = \frac{\sigma}{\sqrt{n}} \]  \hspace{1cm} (A.6)

where;

\( \sigma_x \) is the standard error of the mean for an infinite population

\( \sigma \) is the standard deviation of the population

\( n \) is the number of data
Appendix 2

X - RD RESULTS
Filling materials play a significant role in the behaviour of rock masses particularly in conjunction with the stability of rock slope faces in open pit mines. Composition of infilling materials are very different with different mechanical behaviour. The thickness of infilling materials also has a great effect on the shear strength of the naturally jointed samples and rock masses.

In this thesis, because of importance and presence of different types of infilling materials in different quarries, several samples from the slope faces of the quarries were collected and tested at the Department of Geology using X-RD method. X-ry diffraction is a suited method for the examination of fine-grained of all samples, particularly for identification of infilling materials. Description of details of X-ry method is beyond the scope of this thesis and it could be found in related books and also in the various reports published by the X-ry units in manufactories.

For the purpose of this thesis, as described in chapters 5, 6, and 7, the types and nature of infilling materials were identified during the field investigations by visual examination whether sandy, silty, or clayey materials. In addition the state of infilling materials was indentified during the field investigations by examining the degree of alteration field. Finally the types of infilling materials were identified using X-RD method. The results of X-RD tests are given in the form of X-ry diffractograms for different samples of quarries. The results of X-RD tests of infilling material samples from different parts of Mugga II quarry are presented in Figures A.1 to A.10 and the results of X-RD examinations of infilling materials from different parts of Dunmore Quarry are also presented in Figures A.11 to A.16. Based on the results of X-RD examinations most important of infilling materials in these quarries are calcite and quartz accompany with some very rare minerals resulted by the alteration processes.
Figure A.1  X-ray diffractogram of infilling materials from section A of Mugga II Quarry (Q = Quartz; A = Albite; K = Potassium Gallium Silicate).

Figure A.2  X-ray diffractogram of infilling materials from section A of Mugga II Quarry (Q = Quartz; A = Albite; K = Potassium Gallium Silicate.)
Figure A.3  X-ray diffractogram of infilling materials from section B of Mugga II Quarry (Q = Quartz; A = Albite; K = Potassium Gallium Silicate.

Figure A.4  X-ray diffractogram of infilling materials from section B of Mugga II Quarry (Q = Quartz; Ga = Gallium Phosphate; A = Albite.
Figure A.5 X-ray diffractogram of infilling materials from section B of Mugga II Quarry (Q = Quartz; A = Ammonium Nickel Selenate; A* = Albite.

Figure A.6 X-ray diffractogram of infilling materials from section C of Mugga II Quarry (S = Stellerite; B = Barrerite; S = Stellerite; Q = Quartz; R = Regersite.)
Figure A.7  X-ray diffractogram of infilling materials from section C of Mugga II Quarry (Q = Quartz; Ga = Gallium Phosphate; K = Potassium Gallium Silicate.

Figure A.8  X-ray diffractogram of infilling materials from section C of Mugga II Quarry (Q = Quartz; Ba = Barium Copper Phosphate.
Figure A.9 X-ray diffractogram of infilling materials from section D of Mugga II Quarry (Ca = Calcite; RU = Rubidium Nitrate; Q = Quartz).

Figure A.10 X-ray diffractogram of infilling materials from section D of Mugga II Quarry (Q = Quartz; B = Boron Phosphate; Ga = Gallium Phosphate.)
Figure A.11 X-ray diffractogram of infilling materials from section A of Dunmore Quarry (Q = Quartz; Ga = Gallium Phosphate; Zn = Zinc Ammine.

Figure A.12 X-ray diffractogram of infilling materials from section A of Dunmore Quarry (A = Albite; Mn = Manganese Phosphate; Or = Orthoclase.
Figure A.13 X-ray diffractogram of infilling materials from section A of Dunmore Quarry (St = Stellerite; Ba = Barrerite; Sti = Stilbite; Cl = Clinoptilolite.

Figure A.14 X-ray diffractogram of infilling materials from section B of Dunmore Quarry (Ca = Calcite; Zn = Zinc Oxide.
Figure A.15 X-ray diffractogram of infilling materials from section B of Dunmore Quarry (La = Laumontite; Ne = Newberyite; De = Deuterium Oxide)

Figure A.16 X-ray diffractogram of infilling materials from section B of Dunmore Quarry (Ca = Calcite; La = Laumontite; Ne = Newberyite.)