CONCEPTS USED TO ANALYZE AND DETERMINE ROCK SLOPE STABILITY FOR MINING & CIVIL ENGINEERING APPLICATIONS

by

Scott Daniel Ureel

A Dissertation Submitted to the Faculty of the
DEPARTMENT OF MINING, GEOLOGICAL & GEOPHYSICAL ENGINEERING

In Partial Fulfillment of the Requirements
For the Degree of

DOCTOR OF PHILOSOPHY

In the Graduate College

THE UNIVERSITY OF ARIZONA

2014
THE UNIVERSITY OF ARIZONA
GRADUATE COLLEGE

As members of the Dissertation Committee, we certify that we have read the dissertation

prepared by Scott D. Ureel

entitled CONCEPTS USED TO ANALYZE AND DETERMINE ROCK SLOPE STABILITY FOR MINING & CIVIL ENGINEERING APPLICATIONS

and recommend that it be accepted as fulfilling the dissertation requirement for the Degree of Doctor of Philosophy

Date: 12/17/13
Dr. Moe Momayez

Date: 12/17/13
Dr. John Kemeny

Date: 12/17/13
Dr. Victor Tenorio

Date: 12/17/13
Dr. Lianyang Zhang

Final approval and acceptance of this dissertation is contingent upon the candidate’s submission of the final copies of the dissertation to the Graduate College.

I hereby certify that I have read this dissertation prepared under my direction and recommend that it be accepted as fulfilling the dissertation requirement.

_____________________________ Date: 12/17/13
Dissertation Director: Dr. Moe Momayez
STATEMENT BY AUTHOR

This dissertation has been submitted in partial fulfillment of requirements for an advanced degree at the University of Arizona and is deposited in the University Library to be made available to borrowers under rules of the Library.

Brief quotations from this dissertation are allowable without special permission, provided that accurate acknowledgment of source is made. Requests for permission for extended quotation from or reproduction of this manuscript in whole or in part may be granted by the author.

SIGNED: Scott D. Ureel
ACKNOWLEDGEMENTS

I would like to express great thanks to Dr. Moe Momayez who supported me and offer priceless insight during the work of this dissertation. He kept me on track and helped me succeed through my graduate studies at the University of Arizona. Without his help and knowledge, this dissertation would not have been successfully completed. Also, I would like to acknowledge all the individuals within the University of Arizona Mining & Geological Engineering and Civil Engineering departments most notably Dr. John Kemeny, Dr. Ted Wilson and Dr. Lianyang Zhang. Last year, Dr. Wilson passed away while enjoying his other passion of flying. I am proud to say Ted was a phenomenal individual and thank him for all the time talking with me about mining and his experiences. Dr. Victor Tenorio accepted the role as my committee member and has been extremely helpful in the success of this dissertation. Lastly, I thank my family (Mom, DBU, Moll, Bill, Joeys) for motivating me to the finish!

Some of the text of this dissertation includes reprints of the following previously published and presented material:

# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIST OF FIGURES</td>
<td>8</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>11</td>
</tr>
<tr>
<td>ABSTRACT</td>
<td>12</td>
</tr>
<tr>
<td>1.0 INTRODUCTION</td>
<td>15</td>
</tr>
<tr>
<td>1.1 Rock Core Orientation</td>
<td>16</td>
</tr>
<tr>
<td>1.2 Material Testing &amp; Rock Abrasion</td>
<td>18</td>
</tr>
<tr>
<td>1.3 Slope Stability Modeling</td>
<td>19</td>
</tr>
<tr>
<td>1.4 Slope Stability Monitoring</td>
<td>21</td>
</tr>
<tr>
<td>1.5 Research Objectives</td>
<td>22</td>
</tr>
<tr>
<td>1.6 Research Contributions</td>
<td>23</td>
</tr>
<tr>
<td>1.7 Format of Dissertation</td>
<td>24</td>
</tr>
<tr>
<td>2.0 ROCK CORE ORIENTATION &amp; LOGGING</td>
<td>25</td>
</tr>
<tr>
<td>2.1 Introduction</td>
<td>25</td>
</tr>
<tr>
<td>2.2 Rock Orientation</td>
<td>26</td>
</tr>
<tr>
<td>2.3 Clay Imprint Apparatus</td>
<td>28</td>
</tr>
<tr>
<td>2.4 Ezy-Mark Orientation</td>
<td>30</td>
</tr>
<tr>
<td>2.5 Reflex ACT Apparatus</td>
<td>31</td>
</tr>
<tr>
<td>2.6 Telviewer</td>
<td>33</td>
</tr>
<tr>
<td>2.7 Limitations and Benefits</td>
<td>41</td>
</tr>
<tr>
<td>2.7.1 Clay Imprint</td>
<td>41</td>
</tr>
<tr>
<td>2.7.2 Ezy-Mark Orientation</td>
<td>42</td>
</tr>
<tr>
<td>2.7.3 Reflex ACT I, II, III</td>
<td>42</td>
</tr>
<tr>
<td>2.7.4 Telviewer Orientation</td>
<td>43</td>
</tr>
<tr>
<td>2.8 Conclusion</td>
<td>44</td>
</tr>
<tr>
<td>3.0 MATERIAL TESTING &amp; ROCK PROPERTIES</td>
<td>46</td>
</tr>
<tr>
<td>3.1 Uniaxial Compression Testing (UCS)</td>
<td>49</td>
</tr>
<tr>
<td>3.2 Disk Tension (Brazilian) Testing (DT)</td>
<td>51</td>
</tr>
<tr>
<td>3.3 Triaxial Testing</td>
<td>53</td>
</tr>
<tr>
<td>3.4 Small Scale Direct Shear Testing (SSDS)</td>
<td>54</td>
</tr>
<tr>
<td>3.5 Hardness &amp; Rock Quality Designation (RQD)</td>
<td>55</td>
</tr>
<tr>
<td>3.6 LA Abrasion Testing</td>
<td>57</td>
</tr>
<tr>
<td>4.0 ROCK ABRASION AND ROCK STRENGTH</td>
<td>59</td>
</tr>
<tr>
<td>4.1 Introduction</td>
<td>59</td>
</tr>
<tr>
<td>4.2 Rock Abrasion Experimental Literature</td>
<td>60</td>
</tr>
<tr>
<td>4.2.1 Annandale Erosion Model</td>
<td>61</td>
</tr>
<tr>
<td>4.2.2 Briaud &amp; Associates Apparatus</td>
<td>63</td>
</tr>
<tr>
<td>4.2.3 Scour Model</td>
<td>65</td>
</tr>
</tbody>
</table>
4.3 Rock Abrasion Testing ........................................................................................................ 67
  4.3.1 LA Abrasion and Slake Durability Testing ................................................................ 68
  4.3.2 Nordic Ball Mill Test and Micro-Deval Test ................................................................ 70
  4.3.3 Gouging Abrasion ......................................................................................................... 71
  4.3.4 Abrasion Resistance Hardness Testing ......................................................................... 72
  4.3.5 Concrete Abrasion Apparatus ..................................................................................... 74
  4.3.6 Miller Slurry Testing .................................................................................................... 76
  4.3.7 Rock Abrasion Tool (RAT) .......................................................................................... 77

4.4 Experimental Methods ....................................................................................................... 78

4.5 Results ................................................................................................................................ 79

4.6 Discussion ............................................................................................................................. 89
  4.6.1 UCS, Rock Abrasion and RQD .................................................................................. 89
  4.6.2 UCS, Rock Abrasion, Rock Mass Rating (RMR) and Geological Strength Index (GSI) ................................................................................................................. 91
  4.6.3 Shear Strength, Rock Abrasion and RQD ................................................................. 91
  4.6.4 Summary of Rock Properties Correlations .............................................................. 92
  4.6.5 Summary of Abrasion Testing Characteristics and Future Research ....................... 95

4.7 Conclusion ........................................................................................................................... 100

5.0 SLOPE STABILITY MODELING ...................................................................................... 102
  5.1 Introduction ....................................................................................................................... 102

5.2 Modeling Techniques ......................................................................................................... 104
  5.2.1 Limit Equilibrium Method ......................................................................................... 105
  5.2.2 Shear Strength Reduction Method ............................................................................. 106

5.3 Past Comparisons of Limit Equilibrium vs. Numerical Modeling ................................ 107

5.4 Limit Equilibrium vs. Discrete Element Method ............................................................ 110

5.5 Limit Equilibrium Method vs. Finite Difference Method ............................................... 111

5.6 Continuum Modeling vs. Discontinuum Modeling .......................................................... 115

5.7 Future Research ................................................................................................................. 118

5.8 Conclusion ........................................................................................................................... 122

6.0 SLOPE STABILITY MODELING ...................................................................................... 124
  6.1 Introduction ....................................................................................................................... 124

6.2 Methodology ...................................................................................................................... 125
  6.2.1 Limit Equilibrium Method (LEM) ............................................................................ 126
  6.2.2 Shear Strength Reduction Technique (SSR) ........................................................... 127

6.3 Homogeneous Soil Slope .................................................................................................. 128
  6.3.1 Material Properties ................................................................................................. 129
  6.3.2 Factor of Safety Results ........................................................................................... 130

6.4 Homogeneous Rock Slope ................................................................................................ 131
  6.4.1 Material Properties ................................................................................................. 132
  6.4.2 Factor of Safety Results ........................................................................................... 133

6.5 Heterogeneous Rock Slope ............................................................................................... 134
  6.5.1 Three Material Model ............................................................................................. 134
  6.5.1.1 Material Properties ........................................................................................... 135
  6.5.1.2 Factor of Safety Results ................................................................................... 136
LIST OF FIGURES

Figure 1.1: Flow chart to achieve rock slope stability.................................................17

Figure 2.1: Flow chart step 1 for rock slope stability..................................................26
Figure 2.2: Goniometer used for core orientation.........................................................27
Figure 2.3: Basic visual of reference line, alpha and beta angles
(Cylwik et. al 2011) ..............................................................................................28
Figure 2.4: Clay orientor concept prior and during imprint (Call & Nicholas, Inc.
2008) ..................................................................................................................29
Figure 2.5: Determining top of hole marking by matching true
orientation............................................................................................................30
Figure 2.6: Ezy-Mark system.........................................................................................31
Figure 2.7: Transferring orientation to rock core using Ezy-Mark system..................31
Figure 2.8: Reflex ACT II orientation tools (Reflex Instruments, 2013) ....................33
Figure 2.9: Using the Reflex ACT II tool to find orientation and reset timer.............33
Figure 2.10: Projection of an inclined planar feature intersecting a cylinder and
“unfolding” of the ellipse to 2D results in a fixed-period sinusoid. (Gaillot
et al., 2007) .....................................................................................................36
Figure 2.11A: Acoustic borehole image of Fe-oxidized granite. .................................38
Figure 2.11B: Optical borehole image of Fe-oxidized granite. ....................................38
Figure 2.12A: Acoustic borehole image of relatively fresh monzonite with quartz-
sericite-pyrite veins and potentially open sericite/Fe-oxide-filled
discontinuities......................................................................................................39
Figure 2.12B: Acoustic borehole image of heavily sericitized/argillized monzonite...40

Figure 3.1: Step 2 in flow chart for rock slope stability..........................................46
Figure 3.2: General failure stress vs. confining stress from intact testing...............48
Figure 3.3: General shear strength criterion..............................................................49
Figure 3.4: Uniaxial compression test sample..............................................................50
Figure 3.5: Uniaxial compression test..........................................................................51
Figure 3.6: A disk tension test sample..........................................................................52
Figure 3.7: Disk tension testing apparatus.....................................................................52
Figure 3.8: Triaxial testing apparatus surrounded with σ3 pressure produced by oil..53
Figure 3.9: Small scale direct shear apparatus............................................................55
Figure 3.10: RQD procedure and calculations (Deere et al, 1967) .............................57
Figure 3.11: Steel spheres used in LA Abrasion steel cage........................................58
Figure 3.12: Basic LA Abrasion test ...........................................................................58

Figure 4.1: Observed scour beneath a bridge footing due to flooding conditions ......60
Figure 4.2: Conceptual model showing the process of scour
(after Annandale 1995) .......................................................................................62
Figure 4.3: Erodibility threshold for earth materials (Annandale 1995) .................63
Figure 4.4: Forces applied to soil grains during scour (Briaud, et al, 2001) .............64
Figure 4.5: Correlation between critical shear stress and initial erodibility (after
Figure 4.6: Design example for estimating average erosion (after Dickenson & Baillie, 1999) ................................................................. 67
Figure 4.7: Slake and abrasion test data and conceptual drawing of the experiment set up ................................................................. 69
Figure 4.8: Basic LA Abrasion test and data (Kahraman and Fener, 2007) ............. 70
Figure 4.9: Gouging Abrasion Test (Golovanevskiy & Bearman, 2007) ................. 72
Figure 4.10: ARHT apparatus (Conca & Cubba, 1986) ........................................ 74
Figure 4.11: ARHT example data (Conca & Cubba, 1986) .................................... 74
Figure 4.12: Concrete abrasion testing apparatus (White Machine, Inc. Ohio USA) ........................................................................ 75
Figure 4.13: Example of results produced by concrete abrasion apparatus (Laplante et al, 1991) ................................................................. 76
Figure 4.14: Typical Miller Slurry Abrasion Test apparatus (WRES, Inc. 2013) ..... 77
Figure 4.15: RAT schematic diagram (Thomson et al, 2013) ............................... 78
Figure 4.16: RQD vs. abrasion loss ................................................................ 81
Figure 4.17: GSI rating system (Hoek & Brown, 1997) ....................................... 82
Figure 4.18: GSI vs. abrasion loss for metamorphic rock (linear trend) ............... 84
Figure 4.19: RMR vs. abrasion loss for metamorphic rock .................................. 86
Figure 4.20: Shear strength vs. LA Abrasion loss for metamorphic rock .......... 88
Figure 4.21: RQD vs shear strength for metamorphic rock (logarithmic trend) .... 88
Figure 4.22: Comparison of RQD vs UCS for metamorphic rock ..................... 90
Figure 4.23: Estimating of RQD vs. LA Abrasion loss with extra data points ....... 95
Figure 4.24: Example of possible correlations between RQD, depth of wear, UCS 98
Figure 4.25: Shear force vs. horizontal displacement ....................................... 100

Figure 5.1: Step 3 in the rock slope stability flowchart ..................................... 103
Figure 5.2: Factor of safety comparison between the LEM (FS) and the SSR (FS1), Cala et al (2003) ........................................................................ 109
Figure 5.3: Attributes of the four classes of the discrete element method and the limit equilibrium method (Cundall and Hart 1989) ...................... 111
Figure 5.4: Factor of safety and slip surfaces for 170 m soft rock slope (Cala & Flisiak, 2001) ................................................................. 113
Figure 5.5: Slope failure simulated in PFC2D (Wang et al 2003) ....................... 121

Figure 6.1: 10 meter high soil slope, FOS = 1.0 (LEM) .................................... 128
Figure 6.2: 10 meter high soil slope, FOS = 1.1 (SSR) ...................................... 129
Figure 6.3: 915 meter high rock slope, FOS = 1.0 ......................................... 131
Figure 6.4: 915 meter high, dry rock slope, FOS = 1.1 (SSR) ....................... 132
Figure 6.5: Three material model, FOS = 1.0 ................................................. 135
Figure 6.6: Five rock material model, FOS = 1.0 ............................................. 137
Figure 6.7: 915 m high, dry five rock type material model, FOS = 1.2 (SSR) .... 138
Figure 6.8: Stress conditions for 10 m homogeneous soil slope, FOS = 1.1 ....... 145
Figure 6.9: Stress conditions for 915 m homogeneous rock slope, FOS = 1.1 .... 145

Figure 7.1: Step 4 in the rock slope stability flow chart .................................... 148
Figure 7.2: GIS approach for slope stability analysis (after M.Cai et.al. 2007) ... 151
Figure 7.3: Hyperspectral imaging.................................................................155
Figure 7.4: Classification and mapping of different gypsiferous soil types (after Goossens, R. and Van Ranst, E 1998) ........................................158
Figure 7.5: Left: Ground-Based InSAR (Noferini et al 2006), Right: DEM Geometry.............................................................163
Figure 7.6: Integrated Slope Stability System.................................................166
Figure 7.7: Walnut Gulch Watershed and rain gauges.................................168
Figure 7.8: SRTM image for the study area (NASA, 2000) .........................169
Figure 7.9: TWI map for the study area..........................................................170
Figure 7.10: Variation in the variables and modeling (semi-variogram and cross-semi-variogram) ..............................................................171
Figure 7.11: Soil moisture map for the study area........................................172
Figure 7.12: Slope stability index map for the study area..............................174
LIST OF TABLES

Table 2.1: Benefits and limitation for rock orientation ........................................41

Table 3.1: Relationship between hardness, consistency and UCS (after Call & Nicholas, 2008) ..........................................................56

Table 4.1: Comparison between LA Abrasion, Micro-Deval and Nordic Ball Test...71
Table 4.2: Results of material testing on metamorphic rock (Quartzite) ...............80
Table 4.3: GSI and associated rock properties .....................................................83
Table 4.4: RMR system (after Bieniawski, 1989) .................................................85
Table 4.5: RMR for selected rock samples............................................................86
Table 4.6: Shear strength results...........................................................................87
Table 4.7: Correlation coefficients for original data..............................................92
Table 4.8: Correlation coefficients for adjusted data.............................................93
Table 4.9: Summary of regression calculations and data set...............................93
Table 4.10: Summary of abrasion testing methods..............................................96

Table 5.1: Conventional methods for slope stability (after Stead, et al, 2001) ....105
Table 5.2: Comparison of numerical model solutions to LEM solutions (Wyllie & Mah 2004) .................................................................108
Table 5.3: Numerical methods for slope stability analysis (Coggan, et al, 1998) ...115

Table 6.1: Derived material strengths for 10 meter slope....................................130
Table 6.2: FOS comparison for 10 meter slope....................................................130
Table 6.3: Derived material strengths for 915 meter high slope........................133
Table 6.4: FOS comparison for 915 meter slope................................................133
Table 6.5: Derived material strengths for three material model........................135
Table 6.6: FOS comparison for three material model .........................................136
Table 6.7: Derived material strengths for five material model ............................139
Table 6.8: FOS comparison for five material model ...........................................141
Table 6.9: FOS comparison for 10 meter slope..................................................142
Table 6.10: FOS comparison for 915 meter slope.............................................143

Table 7.1: Remote sensing based products.......................................................157
ABSTRACT

Slope stability plays an important role in rock engineering. During the design, construction and post design phases of rock slope stability, engineers and geologists need to pay close attention to the rock conditions within the rock slope to prevent slope failures, protect employees and maintain economic profit. This dissertation is based on a general four step procedure to construct and maintain rock slope stability with confidence. These four steps include field investigations, material testing and rock strength database, slope modelling and slope monitoring. The author provides past, present and alternatives methods for each step for the introduced slope stability procedure. Specific topics within each step are investigated displaying results, recommendations and conclusions.

Step one involves data collection during field investigations for rock slope design. Orientation of rock core during drilling programs has become extremely pertinent and important for slope stability and underground mining operations. Orientation is needed to provide essential data to describe the structure and properties of discontinuities encountered during the design process to understand favourable and unfavourable conditions within a rock slope and underground openings. This chapter examines and discusses the limitations and benefits of four methods of obtaining borehole discontinuity orientations from drilling programs including clay-imprint, ACT I, II, III Reflex, EZY-MARK, and OBI/ABI Televiewer systems. Results, recommendations and conclusions are provided in this study.

During step two to maintain rock slope stability, a rock strength database was created and used to correlate and compare RQD values to rock abrasion, shear
strength and other rock characterization methods. Rock abrasion plays a significant role in geotechnical design, tunneling operations and the safety of foundations from scour; however, rock abrasion can be used to develop higher confidence in important parameters such as RQD and hardness. More rock abrasivity research is needed to provide a more accurate and compatible method for all subsurface material properties used in mining and civil engineering projects. This report will provide simple correlations relating abrasion resistance to RQD, UCS, Geological Strength Index (GSI) and Rock Mass Rating (RMR) of metamorphic rock. Results, discussions and conclusions are provided.

Step 3 to determine rock slope stability entails utilizing computer modeling to predict failure conditions and wear rock mass properties. Computer modeling and slope monitoring for rock slopes have become essential to assess factor of safety (FOS) values to predict slope instability and estimate potential failure. When utilizing computer models, the limit equilibrium method (LEM) provides FOS values according to force and moment equilibrium; the shear strength reduction (SSR) technique calculates FOS using stress- and deformation-based analyses. Currently, both methods are prevalent in the engineering industry and applied by geotechnical engineers to analyze and determine stability in rock slopes for mining and civil engineering projects. Slope modeling techniques are then used to observe slope conditions and predict when slope failure may occur (FOS = 1.0). Comparison, results and conclusions are presented.

Lastly, the dissertation (step 4: slope monitoring) will investigate past studies of FOS comparisons, review calculation methods and provide procedures and results
using remote sensing data. The main objective of the dissertation is to provide engineers with essential information needed to ensure high confidence in factor of safety predictions and how alternative methods can be utilized. Recommendations, future research and conclusions regarding FOS and slope monitoring are provided within the dissertation.
CHAPTER 1: INTRODUCTION

In the mining and civil engineering industries, slope stability analyses have become essential to ensure site safety, maximize ore removal, and limit interruptions to production. Slope geometry, material properties, and in situ stress conditions need to be investigated in order to understand the potential for rock slides, wedge failures, and other instabilities. Without the proper design and monitoring methods, slope failures can cause devastating health, environmental and economic effects. In order to ensure slope stability is achieved, the author conceived a flow chart consisted of the four major divisions used in characterization, design and observation. The flow chart shown in Figure 1.1 shows the critical path to ensure stability and also is the structure of this dissertation. By following the flow chart, the author researched and investigated a key concept for each of the four major divisions (field investigation, material testing and rock strength database, numerical modeling, slope monitoring).

The flow chart may be utilized as simple, useful procedure to follow in practice or research in order to determine stability of rock slopes. Previously published flow charts such as Read & Stacey (2009), Stead et al. (2006) and Eberhardt et al. (2004), present the complexity and specific details involved for rock slope stability rather than a more general process as introduced in this dissertation. Also, Stead et al (2006) and Eberhardt et. al (2004) exhibit failure mechanisms encountered and increasing complexity involved in analysis of rock slopes. The flow chart presented in this dissertation displays a straight forward procedure to analyze the possible conditions and mechanisms provided in the flow charts referenced above.
Some of the text of this dissertation includes reprints of previously published and presented material by the author. The following sections will provide detailed objectives and procedures for the dissertation.
1.1 Rock Core Orientation

The first step in the stability flow chart indicates field investigation. During field investigations, drilling and rock characteristics programs are enforced to obtain
rock samples, data for rock structure, groundwater encountered and condition of material. In order to understand rock behaviour, rock fabric data collected from oriented core, mapping, core logging provides supplemental information for slope stability analyses. Though all this data is required for slope stability analysis, only rock core orientation is discussed in this dissertation. Orientation of rock core during drilling programs has become extremely pertinent and important for slope stability and underground mining operations. Orientation is needed to provide essential data to describe the structure and properties of discontinuities encountered during the design process to understand favourable and unfavourable conditions within a rock slope and underground openings. Chapter 2 of this dissertation examines and discusses the limitations and benefits of four methods of obtaining borehole discontinuity orientations from drilling programs including clay-imprint, ACT I,II,III Reflex, EZY-MARK, and OBI/ABI Televiewer systems. Results, recommendations and conclusions are provided.

1.2 Material Testing & Rock Abrasion

Once the preliminary field investigation is completed, data and samples collected can be tested to construct a detailed rock properties database. Many special material testing techniques are available; however, this dissertation solely investigated rock abrasion. Rock abrasion plays a significant role in geotechnical design, tunneling operations and the safety of foundations from scour. It is imperative to understand how to test the abrasive properties to help determine the amount of scour at foundation locations in order to prevent structural collapse, wear on drilling
tools and help predict unstable rock conditions. Rock abrasion may be correlated with certain rock properties and classification methods. Abrasion is friction between two particles and the intensity of abrasion will depend on the hardness of the material being abraded. Current practice for estimating maximum rock abrasion is based on the Los Angeles abrasion test; however, more research is needed to provide a more accurate and compatible method for all subsurface materials used in mining and civil engineering projects. Chapter 3 explains rock testing procedures, developed intact rock strength criteria and processes used to obtain field data (RQD, hardness). Chapter 4 of this dissertation will review past, current and future methods for determining rock abrasion resistance and the advantages and disadvantages of various rock abrasion techniques. The dissertation introduces recommendations for future rock abrasion techniques and discusses the use of possible rock abrasion correlations for uniaxial compression testing, shear strength and rock quality designation and rock mass classification. This dissertation will provide simple correlations relating abrasion resistance to RQD, UCS, shear strength, geological strength index (GSI), and rock mass rating (RMR) in order to provide a more accurate description of intact and rock mass rock properties.

1.3 Slope Stability Modeling

Once a rock strength data base and material testing has been completed, two dimensional and three dimensional computer models can be constructed to analyze rock slope susceptibility to failure. Chapter 5 presents a review of the methods used to model large scale rocks in practice. The objective of the work is to create a
resource available to geotechnical engineers pertaining to recent results, common programs used in practice, method limitations and recommendations for future use and research of modeling programs. The following methods are included: (1) the distinct element method (discontinuum) using Itasca’s Universal Discrete Element Code, UDEC; (2) the finite difference method (continuum) using Itasca’s Fast LaGrangian Analysis of Continua, FLAC; and (3) the limit equilibrium method using Geostudios: SlopeW. Computer modeling for large-scale rock slopes has become essential to assess factor of safety (FOS) values to predict slope instability and estimate potential failure. The limit equilibrium method (LEM) provides FOS values according to force and moment equilibrium; the shear strength reduction (SSR) technique calculates FOS using stress- and deformation-based analyses. Currently, both methods are used by geotechnical engineers to analyze large-scale rock slopes for mining and civil engineering projects. Both methods can be used simultaneously to check critical slip surface locations and FOS values.

The study in chapter 6 was performed to understand similarities and differences in FOS calculations between the two computer modeling programs for high rock slopes greater than 600 meters. The FOS values obtained indicate a SSR generates results roughly 0.1 lower than the traditional LEM when dealing with homogeneous rock slope geometry. However, when additional geology is included within the rock slope, resulting FOS values between the two methods may be higher or lower depending on the modeling program limits. Several sensitivities were run using different pore pressure conditions, in situ stress ratios, and geologic conditions.
The results generated from the sensitivities, comparisons between numerical methods and the conclusions are presented.

### 1.4 Slope Stability Monitoring

In order to maintain stable slopes in the design and construction latter stages, slope stability monitoring is crucial for the observation and calculation of rock slope movement in mining operations and civil engineering projects. Besides the conventional monitoring methods, advanced ground and airborne techniques are becoming more available and have demonstrated success for being state-of-the-art and non-invasive. Chapter 7 provides information regarding slope stability analysis including numerical modeling and monitoring with the emphasis on advantages and disadvantages of each technique. A summary of data processing and analysis required for slope conditions using satellite, airborne or ground-based technologies is exhibited. The dissertation also introduces an integrated slope monitoring system where all available multi-scale and multi-resolution data are integrated to enhance knowledge of slope and ground conditions in mining operations. Conclusions and recommendations for the integrated slope monitoring system are presented.

After introducing the above integrated slope monitoring system, a study was performed on soil moisture using remote sensing techniques in order to understand how soil moisture affects slope stability. Slope failures and shallow landslides occur due to shallow subsurface flow convergence, soil saturation and shear strength reduction following a heavy rainfall. The semi-arid environment of southern Arizona experiences a significant increase in precipitation during the summer. As a result, the
soil moisture increases the potential for slope failure due to water table fluctuation and pore pressure escalation. In this study, the relationship between soil moisture distribution and topography information (slope angle, topography wetness index) are examined using the topography wetness index, NASA’s Shuttle Radar Topography Mission (SRTM) images and cokriging. Using the Digital Elevation Model (DEM) generated from remote sensing images, the slope stability analysis is performed in a typical semi-arid environment area using data from the Waternut Gulch Experimental Watershed in Tombstone, AZ. Conclusions and future research are provided.

1.5 Research Objectives

The primary objective for the research performed in this dissertation is to ensure and maintain rock slope stability during the design and production stages in the mining and civil engineering industries. The following are specific objectives for this dissertation:

- Introduce a simple procedure to ensure rock slope stability.
- Investigate field methods and measurements used to construct a rock strength and structure database for slope stability analysis (Chapter 2, 3).
- Analyze rock properties for correlations, relationships and alternative methods for RQD and rock abrasion (Chapter 3, 4).
- Research and analyze correct procedures for slope stability modeling using methods prevalent in practice (Chapter 5).
- Understand the differences in factor of safety values between computer modeling programs for rock slope stability analysis (Chapter 6).
• Construct an integrated slope stability monitoring procedure and provide evidence of its use through remote sensing (Chapter 7)

1.6 **Research Contributions**

This dissertation provides several research contributions for the engineering community. The following list highlights the contributions obtained from the research and testing activities undertaken:

- Introduces correct procedures and significant methods used for rock core orientation providing the benefits and limitations of each method.
- Highlights imperative, correlated relationships between uniaxial compression strength, rock abrasion and rock quality designation to improve confidence for measurements and testing values obtained from the laboratory and field.
- Shows decorative, key review points for rock abrasion and recommends future testing procedures.
- Interprets simple correlations between rock abrasion, UCS, RQD, GSI, RMR and shear strength.
- Shows decorative, key review points for slope stability modeling.
- Analyzes differences in factor of safety values between the limit equilibrium method, discrete element modeling and finite difference method.
- Constructs an integrated slope stability monitoring technique and reviews previously used monitoring methods.
- Performs stability analysis on a rock slope in southern Arizona through integration of ground measurements and remote sensing data.
• Provides results, recommendations and conclusions

1.7 Format of Dissertation

As mentioned in the acknowledgments and previous sections, this dissertation was created from a general interest in rock slope stability and investigates key factor from the four significant steps (field, lab, modeling, monitoring) as provide in Figure 1.1 to ensure rock slope stability. Previously published material where the dissertation author was lead author for the published material. The publications used are mentioned in the acknowledgment. The following chapters will provide reviews, new and presently used procedures rock slope stability, results, recommendations and conclusions.
CHAPTER 2: ROCK CORE ORIENTATION & LOGGING

2.1 Introduction

In the mining and civil engineering industries, slope stability is an important consideration for site safety, maximum ore removal and limited interruptions in production. Many aspects of rock slopes need to be investigated such as rock and hydraulic geometry, geological structures, laboratory properties and stress conditions to provide the highest safety potential. The most important properties of rock slopes that dictate optimal slope angles and rock control are the orientation of rock discontinuities or joints. Numerous methods have been introduced to obtain the orientation of rock discontinuities through drilling; however, only three methods are currently widely used in practice and during drilling programs at mine sites throughout the world.

Many times in the mining industry, engineers perform rock slope design using different types of analyses; however, it is essential to utilize oriented core logging to establish baseline geotechnical data to determine planes of weakness within the rock mass at depth. Once core orientation has been achieved, the data can be plotted on stereonets to determine where adversely oriented joint sets may occur. The following chapter (Step 1 in the flow chart) will examine rock core orienting techniques and discuss associated benefits and limitations for applying these methods in the field for rock core orientation.
2.2 Rock Orientation

Core orientation entails recording the orientation of geologic structures in core samples to obtain the in-situ position of discontinuities to determine favorable and unfavorable conditions of rock masses when analyzing the stability of rock slopes. During the orientation process, the in-situ locations of discontinuities are marked on the top or bottom of the core given by the chosen core orientation method (except televiewer imaging). The rock core is assembled together along a leveled edge such as a driller’s split Shelby tube so the reference line can be drawn. Once the reference angle is measured in a goniometer (Figure 2.2), orientation of structures along the core run can be measured using a goniometer (Figure 2.3). The following parameters are important when recording data for each core run:

- Reference Angle
- Dip Angle (Alpha)
- Dip Direction Angle (Beta)
- Rock Type
- Depth
- Alteration
These parameters are extremely important for characterizing joint conditions and expressions as discontinuities generally dictate the mechanical behavior of bench-scale rock masses. The data provide essential information for designing and analyzing critical slip surface paths, slope angles and bench heights. It should be noted, traditional core orienting methods are most beneficial when used on angled drill holes to intercept as many geological structures and rock types as possible that are of geotechnical interest.
When choosing the correct field method for rock orientation, the engineer or geologist needs to be aware of which method is the appropriate choice for varying engineering conditions. Several methods are available and all have inherent limitations and benefits; however, all methods have provided priceless information for mine design. Key considerations when choosing a field orientation method are:

- Accuracy and reliability of in-situ rock orientation data
- Cost
- Interruptions in drilling
- High performance rating (production rate)
- Difficulty in use
- Condition of rock mass

The following section will explain the concept of four popular orientation field methods and how each has made its contribution to rock orientation.

2.3 Clay Imprint Apparatus

The clay-impression method, originally developed by Call & Nicholas, Inc. (CNI), was used to determine the true orientations of fractures from core drilling (Call, 1982). With the use of an inclined holes 40 to 70 degrees from a horizontal reference, the clay-impression method of orienting core allows for the determination of the true orientation of fractures by using an eccentrically weighted orientor (a core barrel half-filled with lead) to take a clay impression of the bottom of the hole. Based on a top-of-the-hole reading obtained from the clay impression, the logged orientation is transferred to the rock core to determine the alpha and beta angles based off a
reference line. The apparent orientations can then be converted to true orientations.

Modeling clay used for impression needs to be packed tightly within the apparatus and needs to extend far enough past the drill bit to make an accurate impression and also unsaturated if possible. If the rock core contains a smooth break at the end of the drill, orientation may not be possible.

The clay imprint method is not difficult to use relative to other methods and is based on simple concepts of core orientation. The cost is almost negligible and is the only method available that makes an actual imprint of the core condition. This method is not suitable for very discontinuous rock as putting the core together can be very difficult and the clay impression can be filled with fines and gravels. Furthermore, extensive drilling programs will result in higher costs as personnel are required on-site during drilling.

A basic concept of the clay imprint is shown in Figure 2.4. Figure 2.5 shows the engineer “matching” clay impression with bottom of the hole.

![Figure 2.4: Clay orientor concept prior and during imprint (Call & Nicholas, Inc. 2008)]
2.4 Ezy-Mark Orientation

The Ezy-Mark is a mechanical orientation tool located at the front of the inner tube that provides an auditable impression of the bottom of the hole before drilling commences and is manufactured by 2iC Australia Pty., Western Australia. The Ezy-Mark core orientation device is inserted into the drill inner tube and then sent down the drill hole. The inner tube is located behind the drill bit and dropped to the core break from the previous run and the instrument is activated making an impression of the core. During this time orientation balls are then locked into place to save the orientation. The drill operator then needs to pull back the instrument and touch the bottom of hole again. The instrument retracts and drilling can continue. Both the orientation tool and core are then brought to the surface to begin core orientation. The core is then matched with the impression made with the orientation screws locked by the orientation balls in the mechanism and an orientation line can be drawn. Figure
2.6 shows the Ezy Mark orientation tools and Figure 2.7 displays the engineer transferring the orientation from the orientation tool to the core.

![Ezy-Mark system](image1.png)

**Figure 2.6: Ezy-Mark system**

![Transferring orientation to rock core using Ezy-Mark system](image2.png)

**Figure 2.7: Transferring orientation to rock core using Ezy-Mark system**

### 2.5 Reflex ACT Apparatus

The Reflex ACT I, II, and III are core orientation devices developed by Reflex Instruments, a division of Imex Limited, with the main office in Perth, Western Australia. Reflex instruments are becoming increasingly popular and are now being
applied worldwide. The Reflex core orientation system is based on recovering the core barrel orientation at the conclusion of a given run. The Reflex orientation tool (Figure) begins the orientation process by inserting the tool in the core barrel using a specially made shoe. The tool records core barrel orientation each minute during a core run. The Reflex sleeve that attaches to the upper drill rod measures the orientation of the top-of-hole using built in accelerometers. Upon completion of a run, the drill string is left undisturbed while the communication tool, which is on the surface, counts down the time to the next reading; after this, the barrel can be withdrawn. On the surface, the tool is inserted into the end of the barrel and the barrel is rotated until the tool indicates that the barrel is in the same up-down position as it was in the hole. The core, barrel, and shoe are then marked using a spirit level to confirm verticality upward. After the liner is split, the top of core marks are transferred along the length of the recovered core. Figure displays the orientation tools and Figure exhibits the ACT II being used in an orienting core program.

The Reflex ACT II is a relatively easy instrument to use once the operator understands how orientation is achieved from the ACT instrument and how the instrument options operate. The ACT I the user needed to utilize a stopwatch instead of the ACT tool recording orientation every minute where ACT II & III the timer is built into the Reflex instrument. Reflex instruments have now introduced the ACT III which contains more capabilities than both the ACT I & II. Figure 2.8 exhibits the Reflex ACT II orientation tool and Figure 2.9 shows driller with the inserted instrument in the core barrel to either match the orientation mark or reset timer for next drill run.
2.6 Televiewer Orientation

Televiewer imaging utilizes optical and acoustic waveforms emitted from a fixed source housed within a probe to map the borehole wall producing a near-continuous down-hole, photographic-like image of the borehole. The orientation of geologic features including fractures, faults, shear zones, bedding planes, sedimentary
features, and veins can be obtained by both optical and acoustic borehole imaging methods (OBI and ABI, respectively).

The OBI probe incorporates a high resolution, high sensitivity CCD digital camera with matching Pentax optics and is used in clear fluid-filled or dry portions of the borehole. Optical imaging devices record contrasting colors of the rock and discontinuities to create a true color photograph of the borehole wall. Mud-filled holes are imaged by a probe outfitted with a sonar transducer that emits ultrasonic pulses at a range of specified intervals that are reflected off a rotating acoustic mirror. The amplitude and travel-time of reflected acoustic signals are measured and recorded simultaneously. Three-armed calipers are not required by ABI tools as the two-way travel-time log effectively represents the borehole diameter while recording any borehole irregularities or breakouts.

Processing and optimization of raw televiewer data into image logs allows the identification and documentation of discontinuities in the surveyed rock mass. Geologic features appear in image logs as fixed-period sinusoidal waveforms displayed from 0° to 360° (Figure 2.10). Note the point tangential to the sinusoids minimum equals the dip direction and dip degree = arctan (h/d) with h = height of the waveform and d = diameter of the cylinder (borehole) in Figure 2.10. Orientation of the image log to geographic north allows calculation of discontinuity orientations with the amplitude and trough of the sinusoids corresponding to the dip degree and dip direction, respectively.
Acoustic borehole imaging is governed by differences in acoustic impedance between the drilling fluid and adjacent rock formation. Acoustic impedance \( Z \) is defined by the following equation:

\[
Z = \rho \ast V
\]

where \( \rho \) = density, and \( V \) = acoustic velocity. Acoustic signals are separated into transmitted and reflected waveforms at the rock/fluid interface (e.g. borehole wall) and the degree of waveform partitioning is directly dependent on the density and acoustic velocity contrast at the interface. The degree of energy partitioning for a wave that hits an interface at normal incidence is defined as the reflection coefficient or impedance mismatch. This is defined by the following equation:

\[
R = \frac{Z_1 - Z_0}{Z_1 + Z_0}
\]

where \( R \) = reflection coefficient and \( Z_0 \) and \( Z_1 \) equal the acoustic impedances of the first and second medium (e.g. drilling fluid and borehole wall), respectively. High impedance mismatches at the fluid/rock interface results in more acoustic energy being partitioned into reflected waveforms; therefore, the transducer receives higher amplitude signatures. Lower impedance mismatches result in more transmitted and less reflected energy at the interface effectively reducing the amount of energy received by the transducer. Amplitudes of reflected waveforms are illustrated on a false-gradational color scheme image log with low and high amplitude signals depicted as cold and hot colors, respectively.
Interpretation of rock fabric orientation data from image logs can be highly subjective and the accuracy and reliability of ABI and OBI rock fabric orientation data are heavily dependent on the quality of the image log, ground/borehole conditions, and experience of the core logger. The quality of image logs may be reduced by numerous factors including improper surveying and data optimization and by rock mass/borehole conditions. Rock mass and borehole conditions that influence image log quality include:

- Style and pervasiveness of alteration.
- Rock and fracture-fill/discontinuity color (OBI only).
- Rock and fracture-fill/discontinuity density contrasts (ABI only).
- Geometry, frequency, filling thickness, spatial relation, and mineralogy of discontinuities.
- Borehole shape, rugosity, and diameter.
- Suspended dust content, fluid turbidity, and wall coatings (OBI only).

Optical and acoustic borehole imaging methods also require multiple steps to create the associated image log prior to discontinuity orientation data interpretation.
Therefore, many instances exist in which errors can be introduced into the data. These errors may be introduced during surveying and data processing and optimization.

As optical borehole image logs represent true color contrasts, differing color combinations of rock and fracture-fill are primary controls in image log quality provided there are good borehole conditions (no coatings, dust, etc). In Figure 2.11A, the two vertical to sub-vertical bands with increased distortion (black arrows) indicate the probe was decentered during surveying. The vertical banding overprints geologic features reducing the quality of the image log introducing difficulties in confidently identifying and tracing sinusoids; partial sinusoids are observed (red arrows) but they cannot be confidently traced. This issue can be resolved by logging with the core present and validating dip degrees with a goniometer. Similarly colored rocks and discontinuities are generally not well represented and difficult to distinguish in image logs. With all other factors equal, color contrast and sinusoid prominence are directly proportional; an increase in contrast generally corresponds to an increase in sinusoid prominence in the image log (Figure 2.11B). The image log contains highly distorted and low-resolution horizontal bands (black arrows) that indicate either a dirty optical lens or improper surveying techniques resulting in lost or poorly recovered data traces. The red arrows indicate potentially open fractures; however, without the core present, it can be extremely difficult to distinguish between open fractures, healed fractures, and veins. Note how darker colored sinusoids are readily apparent in the lighter colored host rock.
Figure 2.11(A). Acoustic borehole image of Fe-oxidized granite.
For ABI, discontinuities typically appear as dark sinusoidal traces on the image log as they have lower impedance mismatches relative to the surrounding rock mass. (Figure 2.12A). However, if the fracture and/or fracture-fill lacks significant impedance mismatches relative to the surrounding rock mass (i.e. clay-filled fractures in a heavily sericitized granite), the fracture and surrounding rock will be illustrated as cold colors on the image log diminishing the core logger’s ability to confidently identify and trace the sinusoid (Figure 2.12B). Even high quality image logs can result in the introduction of erroneous data if logged by unskilled personnel or without the rock core present. As illustrated by Figure 2.12A, numerous sinusoids are
readily apparent but confidently differentiating between measurable open joints, healed fractures, and veins is problematic unless sinusoids in the image log can be successfully correlated to the same feature in the core. Optical and acoustic borehole imaging can be a reliable alternative to other core orienting methods if it is suitable for the project’s needs, is properly managed at each level and is logged with the core present.

Figure 2.12A: Acoustic borehole image of relatively fresh monzonite) with quartz-sericite-pyrite (black arrows) veins and potentially open sericite/Fe-oxide-filled discontinuities

Figure 2.12B: Acoustic borehole image of heavily sercitized/argillized monzonite.
2.7 Benefits and Limitations

The following section provides a discussion summary describing the benefits and limitations for the four methods presented in this chapter. All methods are of great use; however, one may be better suited for weather, cost, workability, rock and drill-hole conditions. Table 1 illustrates a summary of the limitation and benefits for each method. Sections 4.1 to 4.4 will explain each method individually.

**TABLE 2.1: Benefits and Limitation for Rock Orientation**

<table>
<thead>
<tr>
<th>Method</th>
<th>Cost</th>
<th>Difficulty</th>
<th>Benefits</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay Imprint</td>
<td>Low</td>
<td>Low</td>
<td>Creates an exact imprint of core breakage; Works well with HQ and NQ core sizes; Based on simple concepts for core orientation</td>
<td>Cannot be used on vertical holes; Clay needs to be heated in cold regions for workability; Can slow drill production rate; Most accurate of inclinations 40 to 70 deg. From horizontal</td>
</tr>
<tr>
<td>ACT Reflex</td>
<td>Medium</td>
<td>Low to Medium</td>
<td>Can be used on vertical holes (90 deg.); Has many additional options to accompany drilling and data collection; No drilling downtime; Robust design</td>
<td>May be difficult at first to use; ACT I compared to ACT III can be more complex to use; can lose instrument in unstable drill holes</td>
</tr>
<tr>
<td>Ezy-Mark</td>
<td>Medium to High</td>
<td>Low to Medium</td>
<td>Only small interruption in drilling; Has option for recording data to computer; Robust design</td>
<td>Clay on rock core may cause difficulty in marking orientation; Orientation balls can misfire (only occasionally)</td>
</tr>
<tr>
<td>Televiewer</td>
<td>Low to Medium</td>
<td>Medium to High</td>
<td>Potential to obtain orientation data in poor ground conditions (low RQD zones); Can be applied to vertical and inclined holes; Produces a non-continuous photographic-like image of drill hole; Can be used in various borehole sizes (NQ, HQ); Provides caliper and borehole deviation logs</td>
<td>Data collected after drilling is completed; improper data acquisition and collection may result in unreliable orientation data; interpretation of data may be highly subjective; highly altered and broken ground may produce low-quality image logs; multiple steps in data acquisition, processing, and interpretation; Can contain blind zones; Extremely difficult to use in soils and soft sediments</td>
</tr>
</tbody>
</table>

2.7.1 Clay Imprint
The clay imprint has proven very useful since its conception in the early 1970s. It has been used in numerous projects such as the Tazadit Pit in Mauritania, Africa (Call et al, 1982). The clay imprint is the simplest method to use; however, the clay apparatus needs to be specially made and the instrument can only be used on drill holes 40 to 70 degrees from the horizontal. The most unique concept of this method is the impression made of the bottom-of-hole giving the user an actual clay image.

2.7.2 EZY-Mark Orientation

The Ezy-Mark Orientation method provides the most versatile of the four methods. It continuously allows more and more options available for more accurate orientation such as the Verti-Ori and Ori-Block. This method was used on several projects including Freeport MacMoran Grasberg Project in Indonesia. Ezy Mark is retrieve real time data and allow no down drill time. Ezy Mark may not be appropriate for extremely discontinuous rock or rock with a large amount of clay layering.

2.7.3 Reflex ACT I, II, III

The Reflex ACT orientation tools are very robust and made to handle bad weather conditions. The tools are rated to 6,000 psi water resistance and resilience up to 50,000 G’s of force (Reflex Instruments, 2013). The tool was used in extreme conditions at a high altitude mining project in the Peruvian Andes and showed no sign of damage. The Reflex tools become easy to use once the engineer has learned how
the Reflex timer/computer works and provides real time data. The instrument can be lost in an unstable drill hole.

2.7.4 Televiwer Orientation

Many of the issues concerned with accurately interpreting discontinuity orientation data in OBI and ABI image logs associated with OBI and ABI can be resolved by logging with the rock core present. Benefits of logging with the core present include:

- Calculation of core/image log offsets.
- Accurate characterization of fracture fill, alteration styles, and joint condition/expression.
- Geomechanical core sampling for rock strength testing.
- Ability to validate dip angle of low confidence or partial sinusoids using a goniometer.

As core orienting technology continues to advance, methods are continuously developed and improved to provide cheap and reliable discontinuity orientation data. The appropriate application of core orienting techniques under different engineering circumstances can be challenging as scope of work, budget limitations, time constraints, logistics, ground conditions and expected outcomes must all be considered and different methods are more suitable under varying conditions. Optical and acoustic borehole imaging are best applied to more extensive drilling programs on a restricted or scrutinized budget as OBI and ABI are overall less expensive relative to other core orienting methods simply because it requires less man hours.
As data turn-around time is generally slower, it may be more prudent to orient high-priority drill holes (i.e. acquiring rock fabric orientation data for kinematic analyses of a residual-state failure) with a faster method. Conversely, OBI and ABI are more suitable to acquiring data in heavily fractured or broken rock as they do not rely on the ability to confidently piece core together to collect accurate discontinuity orientations.

2.8 Conclusion

The findings in this chapter were used to promote the companies of the orientation methods and help the user determine which method would be advantageous in different mining scenarios. It was not intended to promote one specific product, but to show how each method has its benefits and limitations. Errors and uncertainties introduced by personnel that may result in low-quality image logs or unreliable discontinuity orientation data include:

- Inexperienced or untrained survey operator (OBI/ABI only).
- Improper and/or infrequent maintenance of survey probes (OBI/ABI only).
- Improper data processing and optimization.
- Logging by inexperienced or poorly trained personnel that lack knowledge of the local geology and ground conditions.
- Inconsistent means of data collection by the core logger such as orientation of undesired structures. For example, measurable open joints are the primary structures of interest for slope stability analyses. Orientation of healed
fractures and/or veins may result in inaccurate characterizations of the site’s dominant rock fabric orientations.

- Careless handling of core from drill rod to sleeve.
- Inaccurate transfer of orientation from core (OBI/ABI exclusive).

This chapter was constructed to help engineers, drillers and geologists better understand rock core orientation methods and how to determine which method is most appropriate for varying mining or civil engineering scenarios. The four methods discussed in this chapter have all shown great potential towards obtaining true rock fabric orientations and have assisted in countless engineering projects to identify unstable conditions.
CHAPTER 3. MATERIAL TESTING & ROCK PROPERTIES

A key aspect for obtaining properties of intact rock, fracture strength and rock masses is laboratory and field testing or measurements. Through laboratory testing essential properties can be used to understand rock behavior, make-up and abrasivity. Numerous laboratory properties were gathered in order to provide significant information relating rock hardness, RQD and abrasion. The following section describes the tests performed and the results acquired to create correlations and relationships between the rock properties. Chapter 3 and 4 will cover Step 2 in the rock slope stability flow chart (Figure 3.1).

These laboratory and field measurements include:

- Uniaxial Compression Testing
- Brazilian Disk Tension Testing
- Triaxial Compression Testing
- Small Scale Direct-Shear Testing
- LA Abrasion Testing
- Rock Quality Designation (field)
- Hardness (field)

The stresses obtained from uniaxial compression, triaxial compression and Brazilian disk tension intact testing develop a general strength criterion exhibited in Figure 3.2 and the general shear strength criterion is displayed in Figure 3.3. The shear strength intact rock failure envelope. The formulas for the intact shear strength are:

\[
\sigma_1 = A + \beta \sigma_3
\]

Where \( A \) and \( \beta \) we obtain \( \phi \) and \( C \) for the intact linear shear strength:

\[
C = \frac{A}{2\sqrt{\beta}}
\]

\[
\tan \phi = \frac{\beta - 1}{2\sqrt{\beta}}
\]

The intact linear shear strength curve is defined as:

\[
\tau = \sigma_n \tan \phi + C
\]
Figure 3.2: General Failure Stress vs. Confining Stress from Intact Testing
3.1 Uniaxial Compression Testing

Uniaxial Compression Testing (UCS) is the most widely used laboratory test when testing rock samples for mining and civil engineering purposes. In a uniaxial compression test, a cylinder of drill core is loaded axially without a lateral confining load until the sample fails. A length-to-diameter ratio of 2:1 is considered optimum for the test. Strengths measured using length-to-diameter ratios other than 2:1 must be corrected to 2:1 using empirical relationships. Compressive strength, determined by uniaxial compression testing, is ordinarily assigned as the intact-rock compressive strength, unless the resulting strength is significantly reduced due to failure of the lab specimen along an obvious discontinuity. Uniaxial samples can also be fitted with
strain gauges to measure lateral and axial strains for estimating the elastic properties of the rock: Young’s Modulus (E) and Poisson’s Ratio ($\nu$). Uniaxial test samples are also typically used to measure bulk density values. Testing standards for Uniaxial Compression Test follows ASTM D2938-79. An example of an UCS test sample and the sample in the testing apparatus are shown in Figure 3.4 and Figure 3.5, respectively.

Figure 3.4: Uniaxial Compression Test sample.
3.2 Brazilian Disk Tension Test

In order to further understand the strength of intact rock and create a precise strength criterion, the tensional strength of the intact rock must be obtained. The Brazilian Disk Tension test consists of diametrically loading a disk of drill core until it fails. The diametrical load induces a tensile stress perpendicular to the direction of loading. A specimen cut to a thickness-to-diameter ratio of 0.5:1 is considered optimal. Tensile strengths determined by the Brazilian Disk Tension Test are assigned as the intact rock tensile strength, unless the specimen fails either in a direction other than the direction of loading, or along an obvious discontinuity. When developing a $\sigma_1$ vs. $\sigma_3$ strength criterion for intact, the Brazilian Disk Tension test will provide the tensile along the negative x-axis. The $\sigma_1$ will be zero. Testing standards for Brazilian
Disk Test follows ASTM D3967-81. Figures 3.6 and 3.7 display a DT sample and the sample in the disk tension apparatus, respectively.

Figure 3.6: A disk tension test sample

Figure 3.7: A failed disk tension sample
3.3 Triaxial Compression Testing

Similar to UCS testing, triaxial compression testing is basically a UCS test with confining pressure. In the case of these laboratory triaxial tests, the confining pressures of $\sigma_2$ (in/out of the page) and $\sigma_3$ (horizontal) are assumed to be of the same value. One sample is prepared for the Triaxial Compression Test. It is cut to length, which is 2.5 times the diameter, and then put in the grinding wheel to flatten and smooth the ends. The sample is then put inside a tube of heat shrink with the platens, and the tubing is shrunk around the sample with a heat gun. The sample is then placed inside the testing rig where it is surrounded by oil which exerts $\sigma_3$ pressure. When this pressure reaches a specified point, the sample is then loaded in the $\sigma_1$ direction until failure. The force at failure is recorded. Testing standards for Triaxial Compression Test follow ASTM D2664-80. Figure 3.8 shows a triaxial compression test with the testing rig surrounded with oil.

Figure 3.8: Triaxial testing apparatus surrounded with $\sigma_3$ pressure produced by oil
3.4 Small Scale Direct Shear Testing

The Small Scale Direct Shear Test is used to measure shear loadings upon a sample. Samples are prepared by setting up the two ends of a horizontally broken rock sample into cement blocks that are used to evenly distribute the load upon the samples. The samples are then placed into the testing rig and loaded up with an initial force. The sample is then put under a shear load and the load is recorded. This is repeated under higher compressive loads based on the type of rock and its dimensions. The intact linear shear strength curve is defined as:

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

Where $\tau$ is the shear strength, $\sigma_n$ is the normal stress encountered at depth, $\phi$ is the friction angle of the specimen and $C$ is the cohesion. Figure 3.9 shows a basic small scale direct shear testing apparatus.
3.5 Hardness & Rock Quality Designation

Rock hardness was measured in the field using a geological pick, knife or thumbnail depending on the hardness of the rock. Table X below exhibits the ranges and procedures for measuring the hardness of rock using both visual and based on the UCS of the rock.

Table 3.1: Relationship between Hardness, Consistency and Uniaxial Compressive Strength (after Call & Nicholas, 2008)
RQD was administered during core logging and based on the methods introduced by Deere et. al (1967). The procedure of the measuring RQD is shown in figure 3.10 and the basic equation to determine RQD is:

\[ RQD = \sum_{i=1}^{n} \frac{(\text{Length of piece} > x)_i}{L} \times 100 \]  \hspace{1cm} (1)

Where \( x = 4 \text{ in. or 10 cm.} \), \( L = \text{total length of core} \), \( N = \text{number of pieces greater than} \ x. \)
3.6 LA Abrasion Testing

Abrasion resistance can be estimated using the LA abrasion testing method. The LA abrasion test (ASTM C131-06) was developed to determine the durability of gravel or crushed rock for use in concrete and asphalt pavements. The method was performed using crushed tested samples at 37.5 mm. or smaller using a sample size of roughly 5 kg. The sample is then placed in a rotating steel cage with specified amount of steel spheres (Figure 3.11) that “abrade” the material for a certain amount of revolutions (500 in each test). This method provides an estimation of rock loss and durability; however, it does not have “direct contact” with the rock specimen of
interest which leaves openings for uncertainty. Figure 3.12 below exhibits the set up for the LA abrasion test.

Figure 3.11: Steel spheres used in LA Abrasion steel cage

Figure 3.12: Basic LA Abrasion test
CHAPTER 4: ROCK ABRASION & ROCK STRENGTH

4.1 Introduction

Rock abrasion can be defined as the process of wearing away a surface by friction when particles of sand or small pieces of rock are carried across its surface by a glacier, stream, or the wind. Abrasion characteristics are constantly required to estimate scour potential in bridge foundations (rock and concrete), determine wear of underground construction materials and provide further information regarding material hardness and density.

Abrasion is very useful in determining scour of concrete and rock foundations used for bridge stability above rivers and streams. Scour is “the result of the erosive action of flowing water, excavating and carrying away loose material from the bed and banks of streams” (Annandale, 1995). Scour (shown below in figure 1) is a very important part of design foundations (especially in bridges) because of its natural intent to wash river bottom sediments or rock particle from beneath footings which will most likely cause structural failure. From a census provided from Briaud, et al (2001), one thousand bridges have collapsed over the last 30 years in the United States and 60% of those failures are due to scour. This research exhibits the topic of scour has numerous open ended questions to be answered. Currently, the most applicable technique for evaluating the potential for scour in rock is an empirically based method for predicting the limit at which scour will occur developed by Annandale (1995).
Besides scour potential, rock abrasion may be used to create correlations to Uniaxial Compression Strength (UCS), Rock Quality Designation (RQD), Shear Strength & Joint Stiffness (very significant rock parameters). If these parameters can be correlated with rock abrasion, more knowledge will be obtained to fully understand hardness, rock quality, rock strength and wear of equipment. Rock strengths, rock classification systems and field rock parameters are crucial when analyzing stability of rock slopes. This chapter will investigate several journal papers and methods used for observing, testing and obtaining rock abrasion characteristics. Methods and results from rock abrasion testing, crucial strength rock properties and classifications (RQD, shear strength, GSI, RMR) are presented. The dissertation will also exhibit how future research on rock abrasion could potentially provide helpful correlations to other rock properties.

Figure 4.1: Observed scour beneath a bridge footing due to flooding conditions

4.2 Rock Abrasion Experimental Literature

Before testing can be done to determine if a specimen has high or low abrasion resistance, the process of abrasion needs to be fully understood. Abrasion follows a similar process as scour and erosion. The following section evaluates
several different academic resources (journal papers, reports, case studies) evaluating the process of rock abrasion through alternative methods used for erosion and scour prediction.

4.2.1 Annandale Erosion Model

Determination of rock scour is crucial for bridge foundation design in waterways. Since rock scour depends on rock abrasion resistance, rock abrasive characteristics are required and the process of rock abrasion needs to be fully understood. Annandale (1995) develops a rational method for determining erodibility of earth materials ranging from cohesionless granular soil to weak bedrock. The method is based on correlations between the rate of energy dissipated of flow and the soil erodibility using field observations and stream performance. Annandale developed a conceptual model exhibiting the process of scour and how particles become dismembered from the soil profile of the river or stream bed. Figure 4.2 displays the process of particle disturbance (rock abrasion process) which is broken into three distinct stages: jacking, dislodgement, and displacement.
In his study, he analyzed 150 sites classifying site geology, determining hydraulic characteristics (mostly importantly stream power/energy) and calculating rock size and quality. The correlation between the rate of energy dissipation ($P$) and a material’s resistance ($K_h$) to erosion can be expressed as the function:

$$P = f \times K_h$$

If $P > f \times K_h$, the erodibility threshold is exceeded, and the material will most likely erode (scour) and vice versa. From the data collected at the 150 test sites, Annandale produces rate of power (which can be determined from rate of energy dissipation) values for each site with their respective erodibility values obtained from their abrasion numbers, roughness numbers and particle size. The resulting figure is created and produces the threshold point where potential scour will and will not scour; hence, high or low abrasion resistance. This is shown below in figure 4.3.
4.2.2 Briaud and Associates Erosion Apparatus

Briaud, et al. (2001) involved using an erosion function apparatus (EFA) in order to predict scour rates. One should note, the EFA results produce erosion rate vs. shear stress where the shear stress and eroding process assumptions are based on the applied force diagram below (figure 4.4).
Figure 4.4: Forces applied to soil grains during scour (Briaud, et al, 2001)

Briaud and his associates performed several tests in the EFA and concluded the apparatus has advantages that include it can be site-specific, numerous parameters of interest can be studied (flow velocity, turbulence, etc) and the final results can release scour prediction. The problem with the apparatus is the validity and factors that play a significant role on scour effects. The apparatus has been used to produce erosion results; however, the accuracy has not been tested. An example of an erodibility (initial erodibility vs. shear stress) plot that will be constructed using the EFA is shown below in figure 4.5. This experiment did provide useful information and data regarding soil in a stream; however, rock abrasion and erosion was not investigated. The concepts provided in this chapter may be useful in determining the abrasive processes and measurements in rock.
4.2.3 Scour Model

The report (Dickenson & Baillie, 1999) on scour contained vital information and procedures leading to future research in the topic of scour prediction. Building the base of the report from Annandale (1995) observations, the report was created to publish a report for the Oregon Department of Transportation in regards to a issue concerning predicting scour on weak bedrock in the Oregon Coast Range for eleven different sites containing outcrops of weak bedrock (many the Tyee formation) throughout the Willamette Valley. The research team gathered geotechnical site
characterization parameters by standard geotechnical testing (core drilling, SPT, Shelby tubes) and laboratory testing which includes slake durability, density, unconfined compression test and abrasion tests. Next, hydraulic parameters were determined for each site of interest using USGS stream gages and performing stream data analyses using the HEC-RAS water surface profile program (presently used by ODOT for scour predictions). Once the hydraulic parameters are obtained, the integrated stream power can be calculated. Finally, surveys of stream beds were compared with data gathered in the 1980s with the data collected in 1998 to estimate erosion. With the computed data and resulting geotechnical parameters (abrasion number), a figure was created for the estimation of average erosion. This figure could be used according to rock abrasion by using velocity and mass of the abrading object rather than the power of the stream. This is shown below in figure 4.6.
4.3 Rock Abrasion Testing

Numerous methods for rock abrasion testing have been introduced and utilized to obtain abrasive properties although some are more practical and acceptable. Journal papers, articles and laboratory reports were examined to ensure which method provides the most accurate measurements. The following section will highlight indirect rock contact methods, direct contact methods and a concrete abrasion testing method that could be used on rock.
4.3.1 Slake Durability and LA Abrasion Test

Abrasion resistance can be estimated using two of the most common methods: the slake durability and LA abrasion testing methods. The slake durability testing method is a test for evaluating the wetting and drying effects on the slaking effects of clay bearing rock and siltstones and standardized by the ASTM D4644. The rock is placed in a cage that rotates at 20 rpm for 10 minutes. A weight loss vs. time plot can be constructed to display the amount of rock material lost due to the rock material being “broken apart” by the rotating cage. The weight loss determined from this method is primarily due to abrasion resistance of material and not its slaking characteristics (Dickenson & Baillie, 1999). Also, an abrasion resistance number can be determined using the equation:

\[ \text{Weight Loss} = \beta \times \ln(T) + B \]

where B is the y intercept on the plot, T is the time elapsed and beta is the abrasion number. On the other hand, the LA abrasion test (ASTM C131) was developed to determine the durability of gravel or crushed rock for use in concrete and asphalt pavements.
These methods do provide an estimation of rock loss and durability; however, they do not have “direct contact” with the rock specimen of interest which can lead to uncertainty. Figure 4.7 above exhibits the conceptual set up for the slake durability test and data obtained by the slake durability and LA abrasion test. Kahraman and Fener (2007) performed numerous LA abrasion and Uniaxial compression tests to understand correlations between the two parameters. Similar testing was performed by Kahraman and Gunaydin (2007); however, point load testing and the Schmidt hammer were utilized instead of uniaxial compression testing. Figure 4.8 below
displays the results for LA Abrasion loss vs. Uniaxial Compression testing and the set up for the LA Abrasion test.

4.3.2 Nordic Ball Mill Test and Micro-Deval Test

These two tests are mostly used to determine the abrasion resistance from the wear caused by studded tires; however, the tests are being used more in the tunneling industry to understand the wear of boring cutters. The Nordic ball mill test and the Micro Deval test have many similarities to the LA abrasion test because the tests are performed on crush aggregates rotating in a drum with steel balls and water. The difference between the LA abrasion and the Nordic ball mill and Micro-Deval tests have more revolutions, smaller ball bearings and faster rotation speeds. The Nordic ball mill test also contains steel ribs inside the machine used to lift and then drop the ball bearings and rock samples which separates it from both the LA Abrasion and Micro-Deval test. Table 4.1 below shows the differences and similarities between the Nordic ball mill test, the LA Abrasion test and the Micro-Deval test. Further research

Table 4.1: Comparison between LA Abrasion, Micro-Deval and Nordic Ball Test (after Hunt, 2001)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>LA Abrasion Test</th>
<th>Micro-Deval Test</th>
<th>Nordic Ball Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate Material Size</td>
<td>2.36 to 25 mm (four grading types)</td>
<td>4.75 to 16 mm (three grading types)</td>
<td>11.2 to 16 mm</td>
</tr>
<tr>
<td>Cylinder Size (Inside Diameter)</td>
<td>711 +/- 5 mm</td>
<td>194 +/- 2 mm</td>
<td>206.5 +/- 2 mm</td>
</tr>
<tr>
<td>Inside Cylinder</td>
<td>508 +/- 5 mm</td>
<td>170 +/- 2 mm</td>
<td>335 +/- mm w/three</td>
</tr>
<tr>
<td>Rotation Speed</td>
<td>30 to 33 rpm</td>
<td>100 +/- 5 rpm</td>
<td>90 +/- 3 rpm</td>
</tr>
<tr>
<td>Total Revolutions</td>
<td>500</td>
<td>9500 to 12,000</td>
<td>5400</td>
</tr>
<tr>
<td>Ball Bearing Size</td>
<td>approximately 46.8 mm diameter</td>
<td>9.5 +/- 0.5 mm</td>
<td>15 +0.01/-0.05 mm diameter</td>
</tr>
<tr>
<td>Abrasion Charge</td>
<td>2500 g to 5000g (depending on grading)</td>
<td>5000 g</td>
<td>7000 g</td>
</tr>
</tbody>
</table>

4.3.3 Gouging Abrasion Test

The Gouging Abrasion test (Golovanevskiy & Bearman, 2007) simulates very high stress abrasion under high-energy impact conditions. During this test, a rectangular rock sample is placed into the rock container below a steel wear tool with a sharp point. The rock container can be adjusted horizontally and vertically in order for the steel wear tool to make contact with the surface of the rock specimen. The pendulum then swings across the rock surface to abrade the rock specimen. This procedure has direct contact with the rock sample; however, the test only covers a small surface area on the rock surface and is an expensive testing apparatus. The
design of the test apparatus for Gouging Abrasion testing is shown schematically in Figure 4.9 below.

Figure 4.9: Gouging Abrasion Test (Golovanevskiy & Bearman, 2007)

4.3.4 Abrasion Resistance Hardness Tester (ARHT)

A method of measuring the hardness of materials as manifested by the abrasion resistance was developed in order to compare the hardness of different
materials as they evolve under changing conditions. A portable field instrument was
designed and built to measure changes in the abrasion resistance hardness of geologic
materials, especially those changes resulting from weathering. During the operation
of ARHT, the operator has control of the load, L, and the depth of penetration, D.
Convenient units obtained from the instrument, that discriminate hardness among
different materials, are the abrasion resistance hardness value \((1/4,)\) and the
coefficient of relative hardening for an exterior surface. The surface hardness can be
determined using the following equation:

\[
H_a = \frac{L \times t}{D} \times 10^{-5}
\]

\[
C = \frac{H_a(\text{exterior})}{H_a(\text{interior})}
\]

\(H,\) the abrasion resistance hardness value is in the range of 0 to 100. This is the same
range and values for RQD. Hopefully with more research this relationship of RQD to
\(H\) can be determined to be accurate. Figure 4.10 below displays the ARHT equipment
and figure 4.11 exhibits example data obtained from the ARHT.
4.3.5 Concrete Abrasion Testing Apparatus

Concrete abrasion has been studied in order to determine the concrete’s resistance to degradation scenarios such as road wear, concrete resistance using
different types of rock (aggregate) and chemical surface protection. The concrete abrasion apparatus is a custom made machine (White Machine Inc, Ohio) where square foot by 4 in thick concrete specimens can be abraded by having direct contact with the specimen of large surface area. The machine can be operated at different time intervals and the type of abrading material can be changed to replicate field conditions where abrasion is prevalent. The information obtained from the machine data can be used to determine a correlation between strength and amount of displacement caused by abrading the concrete. This method could potentially be very beneficial if used on rock specimens and not used solely on concrete. The machine also is in accordance with ASTM-C779: Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces. Figure 4.12 displays the concrete abrasion testing apparatus and figure 4.13 exhibits data collected from the testing machine.

Figure 4.12: Concrete abrasion testing apparatus (White Machine, Inc. Ohio USA)
4.3.6 Miller Slurry Test

The Miller slurry abrasion test is used to determine the abrasivity of constructed slurries. From this abrasion test, the Miller number and the slurry abrasion response number (SAR) can be determined. The Miller number is an index to measure the relative abrasivity of slurries. As the Miller number increases the wear of the specimen increases. The SAR number is ranks the relative abrasive response of the slurry. Common water slurries used in the miller slurry abrasion test is aluminum oxide particles. During the Miller slurry test, motor operated rods oscillate along four test samples that are locked in the water slurry filled holder. Though slurry tests use mostly low-stress abrasion conditions, high stress abrasive scenarios can be simulated. High stress testing will be appropriate to use if stresses are need to fracture the rock being tested. A rubber wheel produces low-stress abrasion while a steel wheel produces high-stress abrasion. This would be advantageous option to determine abrasion resistance for slurry pumps, tunnel boring machines, and drilling operations.
Recently, Petrica, et al (2013) used a similar technique as the Miller slurry abrasion test called the slurry steel wheel abrasion test (SSWAT). Figure 4.14 below exhibits a typical Miller slurry abrasion apparatus.

![Figure 4.14: Typical Miller slurry abrasion testing apparatus (WRES, Inc., 2013)](image)

4.3.7 Rock Abrasion Tool (RAT)

The Rock Abrasion Tool (RAT) was developed by Honeybee Robotics, numerous scientists and engineers for Mars exploration and to expose the interior of rock samples to analyze and indentify. It was designed to mimic the effect of a geologists hammer (Myrick et al., 2004). RAT grinds up to 5 mm deep hole with a diameter of 45 mm. The RAT is deployed on an instrument arm and loaded to 10 to 100 N. The grinding wheel rotates up to 3000 rpm on the rock sample surface while revolving around the center axis of the RAT until the desired abrasion depth is reached. This abrasion method directly abrades the rock and its primary actuator can calculate specific grind energy in term of Joules per cubic millimeters which is also considered an index of grindability (Bartlett et al., 2005). Shear strength and hardness may possibly correlate well with specific energy (erodibility) due to the fact that the shear strength is based on how cohesive (interlocking grain strength) the material is
and the amount friction (\( \phi \)) between the rock grains. Though the RAT has many advantages, it is not readily available on the consumer, requires a flat rock surface to perform the test and expensive. A typical schematic of the RAT is shown in Figure 4.15.

![Figure 4.15: The RAT schematic diagram (Thomson et. al., 2013)](image)

4.4 Experimental Methods

The laboratory-testing program this chapter focused on testing a suite of similar rock types from drillholes in close proximity to each other. The rock samples were taken from twelve different boreholes that were drilled at a large mining operation in the Western United States. The prevalent rock type at the mine site is metamorphic quartzite. Even though the mining operation has unlimited rock samples, only a limited number of samples were tested due to time, cost and rock core available. Once the samples were collected they were taken to a rock mechanics laboratory and tested. The following tests were performed on select samples:

- Uniaxial Compression
• Triaxial
• LA Abrasion
• Brazilian Disk Tension

A total of 64 samples were collected from the drillholes. From the samples, 29 disk tension tests, 16 triaxial compression test and 16 uniaxial compression tests were conducted. The RQD and hardness were taken in the field for each sample. Chapter 3 explains all testing and procedures in detail. After the samples were tested for rock strength, the broken sample pieces were crushed to 1 ½". The crushed samples were then used for the LA Abrasion tests according to ASTM C131- Grading A. During this study, no small scale direct shear tests were performed; however, the following equation was utilized in order to determine shear strength of the intact rock samples:

\[ UCS = 2CoTan[\beta] \]

Where Co = cohesion, UCS = uniaxial compression strength, \( \beta \) = slope of the \( \sigma_1 \) vs. \( \sigma_3 \) intact strength criteria = \( 45 + \frac{\theta}{2} \)

4.5 Results

The results from the rock samples, field and abrasion testing were analyzed using different computational methods to understand the correlations between each rock property. Each plot exhibits the correlation equation and the coefficient of determination of the data. For each LA Abrasion test, corresponding UCS, RQD and hardness values were used for the analysis. The values were then plotted to see if a high correlation or relationships exist between the rock properties of interest. Linear,
exponential and power curve fitting techniques were used to determine the highest
correlation through the least squares regression method and visual examination. Table
4.2 shows the results for the material testing.

<table>
<thead>
<tr>
<th>LA Abrasion</th>
<th>UCS  (MPa)</th>
<th>RQD (%)</th>
<th>Hardness</th>
<th>Friction Angle (deg)</th>
<th>Cohesion (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Loss</td>
<td>29</td>
<td>119.04</td>
<td>79</td>
<td>5</td>
<td>50.5</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>49.07</td>
<td>94</td>
<td>3</td>
<td>44.8</td>
</tr>
<tr>
<td></td>
<td>42</td>
<td>28.22</td>
<td>73</td>
<td>3</td>
<td>38.0</td>
</tr>
<tr>
<td></td>
<td>42</td>
<td>23.79</td>
<td>64</td>
<td>2</td>
<td>37.5</td>
</tr>
<tr>
<td></td>
<td>42</td>
<td>77.55</td>
<td>88</td>
<td>4</td>
<td>40.0</td>
</tr>
<tr>
<td></td>
<td>42</td>
<td>48.97</td>
<td>96</td>
<td>4</td>
<td>46.1</td>
</tr>
<tr>
<td></td>
<td>47</td>
<td>51.53</td>
<td>95</td>
<td>3</td>
<td>44.7</td>
</tr>
</tbody>
</table>

Twelve rock strength criteria were constructed from the laboratory testing and
are provided in Appendix A. Due to the amount of material needed and cost of the
LA Abrasion test, only 7 tests were performed. The LA Abrasion results are shown in
Appendix B and all sample results for the rock properties database is in Appendix C.
Figure 4.16 displays the results for the raw RQD vs. LA Abrasion data.
In order to determine if any correlations exist between LA Abrasion resistance and other geomechanical properties used for slope stability, this chapter also examined geological strength (GSI), Rock Mass Rating (RMR) and shear strength. The GSI is a rock rating system created by Hoek & Brown (1997) used to estimate the rock mass strength. GSI is determined by visually observing the rock structure rated from blocky to disintegrated and surface rock conditions rate very poor to very good. From these conditions and reviewing field and laboratory data, a GSI value can be assigned to classify rock by using Figure 4.17. All samples were inspected by an expert geologist and the author and assigned GSI values.
All samples were inspected and assigned GSI values. The summary of the chosen samples is shown in Table 4.3. It should be noted the GSI values are all
estimated. The GSI values were plotted against the abrasion loss percentages and produced a high correlation ($R^2 = 0.86$).

Table 4.3: GSI and associated rock properties

<table>
<thead>
<tr>
<th>GSI</th>
<th>LA Abrasion Loss (%)</th>
<th>UCS (MPa)</th>
<th>RQD (%)</th>
<th>Hardness</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>29</td>
<td>119.1</td>
<td>79</td>
<td>5</td>
</tr>
<tr>
<td>70</td>
<td>35</td>
<td>49.1</td>
<td>94</td>
<td>3</td>
</tr>
<tr>
<td>55</td>
<td>42</td>
<td>28.2</td>
<td>73</td>
<td>3</td>
</tr>
<tr>
<td>50</td>
<td>42</td>
<td>23.8</td>
<td>64</td>
<td>2</td>
</tr>
<tr>
<td>65</td>
<td>42</td>
<td>77.6</td>
<td>88</td>
<td>4</td>
</tr>
<tr>
<td>60</td>
<td>42</td>
<td>49.0</td>
<td>96</td>
<td>4</td>
</tr>
<tr>
<td>45</td>
<td>47</td>
<td>51.5</td>
<td>95</td>
<td>3</td>
</tr>
<tr>
<td>40</td>
<td>56</td>
<td>44.6</td>
<td>74</td>
<td>3</td>
</tr>
</tbody>
</table>
After examining the relationship of GSI and rock abrasion, a similar approach was taken by comparing the Rock Mass Rating (RMR) and rock abrasion. Like the GSI, the RMR also estimates rock mass strength; however, the RMR estimates rock strengths according to six rock conditions rather than rock structure. The RMR is based on UCS, RQD, joint spacing, joint conditions, groundwater conditions and if the orientation is favorable or unfavorable. The RMR classifies rock very poor 0-20, poor 21-40, fair 41-60, good 61-80 and very good 81-100. The orientation rating was omitted to maintain an unbiased RMR value. All samples were classified according to Table 4.4.
Table 4.4: Rock Mass Rating system (after Bieniawski, 1989)

<table>
<thead>
<tr>
<th>A. CLASSIFICATION PARAMETERS AND THEIR RATINGS</th>
<th>Range of values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
<td>10 MPa</td>
</tr>
<tr>
<td>1. Strength of intact rock material</td>
<td>4 - 10 MPa</td>
</tr>
<tr>
<td>Fracture strength index</td>
<td>2 - 4 MPa</td>
</tr>
<tr>
<td>Uniaxial compressive strength</td>
<td>1 - 2 MPa</td>
</tr>
<tr>
<td>Rating</td>
<td>15</td>
</tr>
<tr>
<td>2. Drill core Quality (%)</td>
<td>90% - 100%</td>
</tr>
<tr>
<td>Rating</td>
<td>20</td>
</tr>
<tr>
<td>3. Spacing of discontinuities (0.5 - 2.0 m)</td>
<td>225 MPa - 300 MPa</td>
</tr>
<tr>
<td>Rating</td>
<td>20</td>
</tr>
<tr>
<td>4. Condition of discontinuities (E)</td>
<td>Slightly rough surfaces</td>
</tr>
<tr>
<td>Very rough surfaces</td>
<td>Separation &lt; 1 mm</td>
</tr>
<tr>
<td>Not continuous</td>
<td>Highly weathered walls</td>
</tr>
<tr>
<td>Unweathered wall rock</td>
<td>Soft gouge &gt;5 mm thick</td>
</tr>
<tr>
<td>Rating</td>
<td>20</td>
</tr>
<tr>
<td>5. Groundwater (Joint water pressure) (mm)</td>
<td>None</td>
</tr>
<tr>
<td>Inflow per 10 m</td>
<td>0</td>
</tr>
<tr>
<td>General conditions</td>
<td>0</td>
</tr>
<tr>
<td>Rating</td>
<td>20</td>
</tr>
</tbody>
</table>

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

<table>
<thead>
<tr>
<th>Strike and dip orientations</th>
<th>Very favourable</th>
<th>Favourable</th>
<th>Fair</th>
<th>Unfavourable</th>
<th>Very Unfavourable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnels &amp; mines</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></td>
</tr>
</tbody>
</table>

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

<table>
<thead>
<tr>
<th>Class number</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
</tr>
</thead>
<tbody>
<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>

D. MEANING OF ROCK CLASSES

<table>
<thead>
<tr>
<th>Class number</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>20 yrs</td>
<td>1 year</td>
<td>1 year</td>
<td>5 yrs</td>
<td>5 yrs</td>
</tr>
<tr>
<td>Cohesion of rock mass (kPa)</td>
<td>&gt; 400</td>
<td>300 - 400</td>
<td>200 - 300</td>
<td>100 - 200</td>
<td>&lt; 100</td>
</tr>
<tr>
<td>Fracture angle of rock mass (deg)</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. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS

<table>
<thead>
<tr>
<th>Discontinuity length (persistence)</th>
<th>&lt; 1 m</th>
<th>1 - 3 m</th>
<th>3 - 10 m</th>
<th>10 - 20 m</th>
<th>&gt; 20 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Separation (meters)</td>
<td>None</td>
<td>&lt; 0.1 m</td>
<td>0.1 - 1.0 m</td>
<td>1.0 - 5.0 m</td>
<td>&gt; 5.0 m</td>
</tr>
<tr>
<td>Roughness</td>
<td>Very rough</td>
<td>Rough</td>
<td>Slightly rough</td>
<td>Smooth</td>
<td>Slickensided</td>
</tr>
<tr>
<td>Rating</td>
<td>6</td>
<td>5</td>
<td>4</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>Weathering</td>
<td>Unweathered</td>
<td>Slightly weathered</td>
<td>Moderately weathered</td>
<td>Highly weathered</td>
<td>Decomposed</td>
</tr>
<tr>
<td>Ratings</td>
<td>6</td>
<td>6</td>
<td>3</td>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

<table>
<thead>
<tr>
<th>Strike perpendicular to tunnel axis</th>
<th>Strike parallel to tunnel axis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drive with dip - Dip 45° - 90°</td>
<td>Drive with dip - Dip 20° - 45°</td>
</tr>
<tr>
<td>Favorable</td>
<td>Favorable</td>
</tr>
<tr>
<td>Drive against dip - Dip 45° - 90°</td>
<td>Drive against dip - Dip 20° - 45°</td>
</tr>
<tr>
<td>Very favourable</td>
<td>Very unfavourable</td>
</tr>
<tr>
<td>Fair</td>
<td>Fair</td>
</tr>
</tbody>
</table>

*Drive in direction of dip minus strike. **
The summary of rock samples selected is shown in Table 4.5. It should be noted the RMR values are all estimated. The RMR values were plotted against the abrasion loss percentages and produced a high correlation ($R^2 = 0.87$).

Table 4.5: Rock Mass Rating for selected rock samples

<table>
<thead>
<tr>
<th>UCS Rating</th>
<th>RQD Rating</th>
<th>Joint Spacing Rating</th>
<th>Groundwater Rating</th>
<th>Joint Condition Rating</th>
<th>RMR</th>
<th>Rock Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>17</td>
<td>10</td>
<td>10</td>
<td>30</td>
<td>79</td>
<td>Good</td>
</tr>
<tr>
<td>7</td>
<td>20</td>
<td>15</td>
<td>10</td>
<td>20</td>
<td>72</td>
<td>Good</td>
</tr>
<tr>
<td>4</td>
<td>13</td>
<td>10</td>
<td>10</td>
<td>20</td>
<td>57</td>
<td>Fair</td>
</tr>
<tr>
<td>2</td>
<td>13</td>
<td>8</td>
<td>10</td>
<td>20</td>
<td>53</td>
<td>Fair</td>
</tr>
<tr>
<td>7</td>
<td>17</td>
<td>15</td>
<td>10</td>
<td>10</td>
<td>59</td>
<td>Fair</td>
</tr>
<tr>
<td>4</td>
<td>20</td>
<td>15</td>
<td>10</td>
<td>10</td>
<td>59</td>
<td>Fair</td>
</tr>
<tr>
<td>7</td>
<td>20</td>
<td>15</td>
<td>10</td>
<td>0</td>
<td>52</td>
<td>Fair</td>
</tr>
<tr>
<td>4</td>
<td>13</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>47</td>
<td>Fair</td>
</tr>
</tbody>
</table>

Figure 4.19: RMR vs. LA abrasion loss for metamorphic rock

$y = -1.2606x + 112.54$

$R^2 = 0.8745$
Lastly, shear strengths were determined using the relationship based on the intact strength criteria ($\sigma_1$ vs. $\sigma_3$) and mentioned in the previous section. The Mohr Coulomb shear strength equation was applied using the corresponding sample UCS value as the normal stress applied. Table 4.6 displays the results obtained from the field, laboratory testing and the calculated shear strength.

Table 4.6: Shear Strength Results

<table>
<thead>
<tr>
<th>Rock Abrasion Loss (%)</th>
<th>UCS (MPa)</th>
<th>RQD (%)</th>
<th>Friction Angle (deg)</th>
<th>Cohesion (MPa)</th>
<th>Shear Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>29</td>
<td>119.1</td>
<td>79</td>
<td>50.5</td>
<td>21.4</td>
<td>165.8</td>
</tr>
<tr>
<td>35</td>
<td>49.1</td>
<td>94</td>
<td>44.8</td>
<td>10.2</td>
<td>59.0</td>
</tr>
<tr>
<td>42</td>
<td>28.2</td>
<td>73</td>
<td>38.0</td>
<td>6.9</td>
<td>28.9</td>
</tr>
<tr>
<td>42</td>
<td>23.8</td>
<td>64</td>
<td>37.5</td>
<td>5.9</td>
<td>24.1</td>
</tr>
<tr>
<td>42</td>
<td>77.6</td>
<td>88</td>
<td>40.0</td>
<td>18.1</td>
<td>83.2</td>
</tr>
<tr>
<td>42</td>
<td>49.0</td>
<td>96</td>
<td>46.1</td>
<td>9.8</td>
<td>60.7</td>
</tr>
<tr>
<td>47</td>
<td>51.5</td>
<td>95</td>
<td>44.7</td>
<td>10.7</td>
<td>61.7</td>
</tr>
<tr>
<td>56</td>
<td>44.6</td>
<td>74</td>
<td>45.9</td>
<td>9.0</td>
<td>55.0</td>
</tr>
</tbody>
</table>

The shear strength was then plotted against the corresponding RQD and LA Abrasion values. Figure 4.20 displays the raw data of the shear strength vs. LA Abrasion that produces a low to medium linear correlation with $R^2 = 0.34$. Figure 4.21 shows the results when RQD vs. shear strength values are plotted. This calculates a medium correlation of $R^2 = 0.67$. 
Figure 4.20: Shear strength vs. LA Abrasion for metamorphic rock

\[ y = -148.8 \ln(x) + 620.69 \]
\[ R^2 = 0.4286 \]

Figure 4.21: RQD vs. shear strength for metamorphic rock (Logarithmic trend)

\[ y = 23.144 \ln(x) - 6.7754 \]
\[ R^2 = 0.6463 \]
4.6 Discussion

After the field and laboratory data has been completed, explanation and verification are needed to understand the relationships between RQD, LA Abrasion and other significant rock properties. The following sections will discuss the bias in data, the results obtained from the laboratory and field results and the potential future research needed to further understand the correlations between the mentioned rock properties.

4.6.1 Uniaxial Compression Strength, Rock Abrasion, Rock Quality Designation

Kahraman and Fener (2007) performed numerous LA abrasion and uniaxial compression tests to understand correlations between the two parameters. Similar testing was performed by Kahraman and Gunaydin (2007); however, point load testing and the Schmidt hammer were utilized instead of uniaxial compression testing. Figure 4.19 shows high comparison from Kahraman and Fener (2007) and this study indicating strong relationship between UCS and LA Abrasion and verifies the values obtained.
Though it appears for the constructed plots that no strong correlation exists between RQD and LA abrasion, a large sampling program may increase the correlation due to a larger range of values. The testing and field results maintained a small range of 29 to 56 percent for abrasion loss and 64 to 96 percent for RQD. Both ranges have a difference of about almost 30 percent; however, the ranges lie in different positions among the similar scale of 0 to 100 percent. Theoretically for the relationship of RQD to LA Abrasion to correlate well, the values obtained from rock samples need to have the same indirect ranges and values (ex: LA Abrasion loss = 15 percent, RQD = 85 percent). The differences of the values fall on the 0 – 100 percent scale indicate one or more rock properties is needed to predict RQD instead of RQD.
solely based from rock abrasion resistance. This is similar to if RQD was plotted against UCS except the data displays much scatter. The plots do show some trend; however, it is not conclusive that a correlation exists.

4.6.2 Uniaxial Compression Strength, Rock Abrasion, Rock Mass Rating and Geological Strength Index

Since the GSI and RMR systems include (depending on the rock characterization method used) rock structure, field observations and UCS values, the LA Abrasion data can be used to classify the rock samples accordingly. High correlations exist between RMR and LA Abrasion loss and GSI and La Abrasion loss. It seems GSI applies more to rock abrasion and RQD comparisons than RMR due to GSI being based on rock mass fractures, block size and rock conditions at the surface. However, the RMR characterization does include RQD as a factor when classifying the rock mass. Though the GSI and RMR vs. rock abrasion results show high correlation ($R^2$ above 0.8), the results are biased due to estimation. Both the RMR and GSI have RQD are involved in the classification of the rock mass; however, other factors such as joint conditions, block size and groundwater conditions with help dictate the outcome of the results.

4.6.3 Shear Strength, Rock Abrasion and Rock Quality Designation

From all rock properties, shear strength exhibits the highest correlation with RQD. A medium correlation ($R^2 = 0.65$) RQD vs. shear strength may due to the fact that the shear strength is based on how cohesive (interlocking grain strength) the material is and the amount friction (phi) between the rock grains. One of the main
factors effecting abrasion resistance is the amount of friction between two entities.

The correlation between LA Abrasion and shear strength provide a medium correlation \((R^2 = 0.43)\).

4.6.4 Summary of Rock Properties Correlations

Transformations can be considered once laboratory and field testing is completed and data is compiled. The coefficient of determination \((R^2)\) shown in the plots above indicate how well the data points fit in the statistical model or the measure of predictability of the variance in the regressions. Some plots showed a high coefficient of determination; however, further justification is warranted. The correlation coefficient \((R)\) gives the degree of correlation between two sets of data. The following tables presented show multiple linear regressions created from Matlab with the correlation coefficients provided from the original data and a trial of adjusted data points that included transforming the RQD data to \(1/RQD\). The \(1/RQD\) was used in order to observe if the sign of the abrasion to RQD correlation coefficient changes from positive to negative to obtain correlation coefficients with same signage.

Table 4.7: Correlation coefficients for the original data

<table>
<thead>
<tr>
<th>Abrasion</th>
<th>UCS</th>
<th>RQD</th>
<th>Hardness</th>
<th>Friction Angle</th>
<th>Cohesion</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.6737</td>
<td>0.1179</td>
<td>-0.5638</td>
<td>-0.592</td>
<td>-0.5781</td>
</tr>
<tr>
<td>-0.6737</td>
<td>1</td>
<td>0.213</td>
<td>0.8926</td>
<td>0.7326</td>
<td>0.9764</td>
</tr>
<tr>
<td>0.1179</td>
<td>0.213</td>
<td>1</td>
<td>0.3514</td>
<td>0.5329</td>
<td>0.2261</td>
</tr>
<tr>
<td>-0.5638</td>
<td>0.8926</td>
<td>0.3514</td>
<td>1</td>
<td>0.7292</td>
<td>0.868</td>
</tr>
<tr>
<td>-0.592</td>
<td>0.7326</td>
<td>0.5329</td>
<td>0.7292</td>
<td>1</td>
<td>0.6026</td>
</tr>
<tr>
<td>-0.5781</td>
<td>0.9764</td>
<td>0.2261</td>
<td>0.868</td>
<td>0.6026</td>
<td>1</td>
</tr>
</tbody>
</table>
Table 4.8: Correlation coefficients for the adjusted data

<table>
<thead>
<tr>
<th>Abrasion</th>
<th>UCS</th>
<th>RQD</th>
<th>Hardness</th>
<th>Friction Angle</th>
<th>Cohesion</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0000</td>
<td>-0.6737</td>
<td>-0.0629</td>
<td>-0.5638</td>
<td>-0.5920</td>
<td>-0.5781</td>
</tr>
<tr>
<td>-0.6737</td>
<td>1.0000</td>
<td>-0.2855</td>
<td>0.8926</td>
<td>0.7326</td>
<td>0.9764</td>
</tr>
<tr>
<td>-0.0629</td>
<td>-0.2855</td>
<td>1.0000</td>
<td>-0.4271</td>
<td>-0.5600</td>
<td>-0.3015</td>
</tr>
<tr>
<td>-0.5638</td>
<td>0.8926</td>
<td>-0.4271</td>
<td>1.0000</td>
<td>0.7292</td>
<td>0.8680</td>
</tr>
<tr>
<td>-0.5920</td>
<td>0.7326</td>
<td>-0.5600</td>
<td>0.7292</td>
<td>1.0000</td>
<td>0.6026</td>
</tr>
<tr>
<td>-0.5781</td>
<td>0.9764</td>
<td>-0.3015</td>
<td>0.8680</td>
<td>0.6026</td>
<td>1</td>
</tr>
</tbody>
</table>

A simple linear regression can be constructed using UCS as the independent variable and abrasion as the dependent variable. Next, a multiple linear regression can be constructed and calculated using all variables. Since the correlation coefficient between abrasion and RQD is very low, the RQD variable was taken out of the dataset and the regression was calculated again. Table 4.9 below exhibits the results from the linear and multiple regressions.

Table 4.9 Summary of regression calculations and dataset

<table>
<thead>
<tr>
<th>Abrasion Regression Type</th>
<th>Variables Used</th>
<th>Coefficient of Determination $R^2$</th>
<th>Predicted Abrasion Equation</th>
<th>Correlation Coefficient R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>UCS</td>
<td>0.4539</td>
<td>46.8348 - 0.1227 * UCS</td>
<td>-0.6737</td>
</tr>
<tr>
<td>Multiple</td>
<td>All</td>
<td>0.6424</td>
<td>72.126 - 0.2574 * UCS - 1092.9051 * RQD + 0.0348 * Hardness - 0.3438 * FrictionAngle + 0.8909 * cohesion</td>
<td>-0.8015</td>
</tr>
<tr>
<td>Multiple</td>
<td>All except RQD</td>
<td>0.6372</td>
<td>18.3729 - 0.7031 * UCS + 0.3611 * Hardness + 0.5880 * FrictionAngle + 0.5880 * cohesion</td>
<td>-0.7983</td>
</tr>
</tbody>
</table>
From the dataset analyses, it is shown that abrasion and UCS provide the highest correlation (-0.6737) among all the variables. The multiple regression using all the variables provides an even higher correlation at -0.8015. The results also show that if you do not include the lowest correlation of RQD and abrasion, the correlation coefficient does not increase. This is not what is expected; however, the analysis shows there is possibly a significant correlation between the geomechanical properties and rock abrasion.

The linear and power law relationship still holds when more RQD and Abrasion data points added and indicates a more rigorous relationship between RQD and abrasion. If four data points are added to expand the limited range of LA and RQD values, a correlation of $R^2 = 0.7$ is calculated. This was assuming values one might expect when graphing RQD vs. LA (high RQD = low abrasion loss, low RQD = high abrasion loss). Figure 4.23 below exhibits this. This means further testing is valid and more samples with High RQD (high abrasion resistance) or Low RQD (low abrasion resistance) are needed.
4.6.5 Summary of Abrasion Test Characteristics and Future Research

After investigating the available abrasion testing methods for rock, a summary table can be created to compare the advantages, limitations and parameters that can be correlated to the data obtained by each test. Table 4.10 below gives a summary of four test methods investigated.

Figure 4.23: Estimation of RQD vs. LA Abrasion loss with four extra data points
<table>
<thead>
<tr>
<th>Method</th>
<th>Advantages</th>
<th>Limitations</th>
<th>Parameters that may be Correlated</th>
</tr>
</thead>
<tbody>
<tr>
<td>LA Abrasion Test</td>
<td>• Weight Loss • Crushability Index • Most widely used</td>
<td>• Indirect method • Mostly used on rock aggregate and not rock samples • Only can use crushed rock samples</td>
<td>• RQD • UCS • Shear strength</td>
</tr>
<tr>
<td>Nordic Ball Mill Test &amp; Micro-Deval Test</td>
<td>• Degradation directly between ball and aggregate • Weight Loss</td>
<td>• Indirect method • Used on crushed rock samples (aggregates)</td>
<td>• RQD • UCS • Shear strength</td>
</tr>
<tr>
<td>Gouging Test</td>
<td>• Direct contact with rock specimen • Fast</td>
<td>• Expensive • Only makes contact with small surface area • Hard to control</td>
<td>• RQD • Abrasion # • Deformation</td>
</tr>
<tr>
<td>ARHT Apparatus</td>
<td>• Used in Field • Can control loading and other parameters</td>
<td>• Not for use in Laboratory • Weak Correlations • Only observes field conditions • Large machine to mobilize</td>
<td>• Deformation</td>
</tr>
<tr>
<td>Concrete Abrasion Machine</td>
<td>• Method with abrasion on largest surface area • Most direct method for determining abrasion resistance • Has not been used on rock • Can simulate different abrasion scenarios</td>
<td>• Need to be specially made • Hard to obtain square rock samples</td>
<td>• Joint Stiffness • RQD • Shear Strength • UCS • Deformation</td>
</tr>
</tbody>
</table>
The table provides information indicating which testing methods would be most suitable for testing the abrasion resistance of rock and the rock properties that may be correlated. The LA abrasion and Nordic ball test have been used for numerous years and do provide the weight loss of rock by a rotating cage; however, the tests do have direct contact with the rock samples. An RQD value may possibly be obtained from these tests. The gouging and ARHT tests do provide direct abrading contact with the rock samples but the limitations outweigh the advantages when correlating correlations; however, direct wear can be measured. The more appropriate choice for performing abrasion tests would be the concrete abrasion machine. With this machine, the operator is able to provide a direct abrading contact with the rock sample of interest. The machine also covers or abrades a large surface area compared to other abrasion tests. The Miller slurry test could be used to determine shear strength and joint stiffness of rock; however, this test will not be suitable for direct
abrasion testing. Lastly, the RAT should be used of general, direct rock abrasion in order to understand if certain abrasive correlations to rock properties (shear strength, RQD, UCS) do exist for different rock types. All abrasion testing and methods serve its own purpose; however those purposes need to be explored further with rock core specimens encountered in geotechnical engineering projects.

By knowing the strength of the rock, the UCS and other easily obtained parameters, a correlation may be possible to RQD and joint stiffness. An example plot that can be created is shown in figure 4.24 below. The two parameters predicting the RQD can be substituted with other relevant rock parameters.

![Figure 4.24](image)

**Figure 4.24:** Example for possible correlation of RQD to depth of wear and UCS

From figure 4.24, the depth of wear can be determined from the concrete abrasion testing machine or gouging apparatus on the rock specimen and the UCS of
the rock specimen can be determined by the Uniaxial Compression Strength Test. As expected, the RQD of the rock sample will decrease as the depth wear increases and the UCS is lowered. RQD is based on joint spacing, stiffness and rock strength. Since this idea is in its preliminary stages, no exact conclusion can be made about how the data and apparatuses will behave. The data obtained from direct abrasion methods or the even the Miller slurry test may be able to predict what the joint stiffness of different rock specimens. Example relationships and equations are shown below.

The joint stiffness and shear strength are given by the following equations:

\[ k_n = \frac{\sigma_n}{\Delta \nu} \quad \text{and} \quad k_s = \frac{\tau}{\Delta \nu} \]

where \( k_n \) is the joint normal stiffness, \( k_s \) is the joint shear stiffness, \( \nu \) is the normal displacement, \( u \) is the shear displacement, \( \tau \) is the shear strength and \( \sigma_n \) is the normal stress. Example of shear displacement and shear force is shown in 4.25.
4.7 Conclusions

In conclusion, predicting abrasion resistance and correlating it to other rock parameters such as RQD or joint stiffness can be an intensively time consuming task. The purpose of this chapter was to inform the engineering community of potential future research for rock abrasion and suggest possible methods to obtain reliable data. The following conclusions can made from the research and literature review performed in this chapter:

- New methods for RQD and Abrasion estimates need to be investigated and established to ensure the most accurate values
- Rock abrasion can be studied using apparatuses constructed for different materials than rock.
- Ideas are out there to potentially acquire more accurate values
• Correlations are available; however, many provide inconsistencies or are based on a small database

• Previous scour and erosion investigations and techniques can be applied to rock abrasion

• Apply testing from different fields to measure rock abrasion

• Data from more rock types should be collected

• Results should be determined from direct abrasion data

• Perform abrasion resistance testing to many more specimens rather than only rock aggregates

• Need a method that causes little disturbance to estimate RQD

• RQD is closely related to shear strength including cohesion and friction angle

• Perform periodic surveys and archive data in a easily retrievable manner

• The abrasion process can strictly governed by jacking and dislodgement, dislodgement and abrasion or abrasion only

• Should Investigate all variables effecting abrasion resistance

As technology advances and new ideas are introduced to the engineering community, rock abrasion resistance will become easier and more accurately measured for rock properties. The abrasive properties of rock will hopefully provide valuable information that can be used with future correlations and previous erodibility and rock scour studies.
CHAPTER 5: SLOPE STABILITY MODELING

5.1 Introduction

In the mining and civil engineering industries, slope stability analyses have become essential to ensure site safety, maximum ore removal and limited interruptions in production. Rock and hydraulic geometry, properties and stress conditions need to be investigated in order to determine and predict locations for potential rock slides, wedge failures, planar failures and deep circular failures. For the past few decades, research in slope stability has developed with technology which led to present day utilization of numerical modeling methods. Numerous numerical programs are available to assist geotechnical engineers in several tasks such as estimating the factor of safety (FS) for the stability of the road cut or mine slopes, determining displacements in the rock slope and predict the in-situ stresses present within the rock slope; however, the methods used in these programs will all compute different results and produce individual slip surfaces. Chapter 5 and 6 will investigate Step 3 in the rock slope stability flow chart which includes numerical modeling and the limit equilibrium method (Figure 5.1)
Currently, several different options of numerical methods are available for both continuum and discontinuum methods; however, for purposes of rock slope stability analyses, the Finite Difference Method (FDM), Discrete Element Method (DEM) and the Limit Equilibrium Method (LEM) are most commonly used in practice. Each method contains its own unique process, limitations, advantages and disadvantages for obtaining the critical slip surface and factor of safety. The following chapter will perform a review of slope stability numerical modeling for
rock slopes in open pit mines using LEM, DEM and FDM methods accompanied with literature, past research discoveries, successful strategies and recommendations.

5.2 Modeling Techniques

5.2.1 Limit Equilibrium Method

The Limit Equilibrium Method (LEM) is based on static force and moment equilibrium. The method has been in widespread use since the early 20th century and may be considered the oldest method used for stability calculations in geological engineering. The factor of safety is calculated as a ratio of resisting forces to driving forces and expressed in the following format:

\[
FS = \frac{\text{Resisting Forces}}{\text{Driving Forces}} \quad (1)
\]

\[
FS = \frac{cA + W \cos \psi \tan \phi}{W \sin \phi} \quad (2)
\]

Where; \(c\) is cohesion, \(\psi\) is the angle of the slope, \(\phi\) is the friction angle of the rock material and \(W\) is the weight of the rock mass in question. When the cohesion of the material is not present in equation (2) and the slope is dry, the rock block will slide when the dip angle of the sliding surface equals the friction angle of the rock [9]. Stability is independent of any deformation or the size of the sliding block, and the driving forces equal the resisting forces indicating a FOS of 1.0. A FOS of 1.0 is assumed to be the “threshold” where the rock slope goes from stable to unstable. When the FOS is less than 1.0, the driving forces overcome the resisting forces causing slope movement.

As mentioned in Sjoberg (1999), numerical analyses were not intended to represent specific scenario in extreme detail. The intention of the numerical model is to give the user an idea of the general outline of the rock slope and predicted failures.
Table 1 exhibits the conventional methods for slope stability prior to numerical models were a standard in practice.

Table 5.1: Conventional Methods for Slope Stability (after Stead, et al, 2001)

<table>
<thead>
<tr>
<th>Analysis Method</th>
<th>Critical Input Parameters</th>
<th>Advantages</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stereographic and Kinematic</td>
<td>Critical slope and</td>
<td>Relatively simple to use and give an initial indication of failure potential.</td>
<td>Only really suitable for preliminary design or non-critical slopes. Need to</td>
</tr>
<tr>
<td></td>
<td>discontinuity geometry;</td>
<td>Some methods allow identification and analysis of critical keyblocks. Links</td>
<td>determine critical discontinuities that require engineering judgment. Must be</td>
</tr>
<tr>
<td></td>
<td>representative shear</td>
<td>are possible with other analysis methods. Can be combined with statistical</td>
<td>used with representative discontinuity/joint shear strength data.</td>
</tr>
<tr>
<td></td>
<td>strength calculations</td>
<td>techniques to indicate probability of failure and associated volumes</td>
<td>Primary evaluates critical orientations, neglecting other important joint</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>properties.</td>
</tr>
<tr>
<td>Limit Equilibrium</td>
<td>Representative geometry</td>
<td>Wide variety of software available for different modes (planar, toppling,</td>
<td>Factor of safety calculations give no indication of instability mechanisms.</td>
</tr>
<tr>
<td></td>
<td>and material characteristics; soil or rock mass shear strength parameters (cohesion and friction); discontinuity shear characteristics; groundwater conditions; reinforcement characteristics and external support data</td>
<td>and friction). Mostly deterministic but increased use of probabilistic analysis.</td>
<td>Numerous techniques all with varying assumptions. Strains and intact failure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Can analyze factor of safety sensitivity to changes in slope geometry and</td>
<td>not allowed for. Do not consider in situ stress state. Probabilistic analysis</td>
</tr>
<tr>
<td></td>
<td></td>
<td>material behavior. Capable of modelling 2-D and 3-d slopes with multiple</td>
<td>requires well-defined input data to allow meaningful evaluation. Simple</td>
</tr>
<tr>
<td></td>
<td></td>
<td>materials, reinforcement and groundwater profiles.</td>
<td>probabilistic analyses may not allow for sample/data covariance.</td>
</tr>
</tbody>
</table>

As shown in table 5.1, the LEM can evaluate a wide range of failure modes and contains many vital critical input parameters; however, displacement, velocities and stresses are not calculated. As Sjoberg (1999) stated the LEM is a theory of plasticity and a perfectly plastic material and an associated flow rule must be assumed. This is the crucial difference between analyzing rock slopes with the LEM or stress-based numerical models. The LEM does; however, calculate thousands of factor of safety values in a short amount of time where numerical models can take several hours or even a day depending on the complexity of the geometry and input parameters. The LEM has its limits and numerical methods have their errors. Zhang & Sanderson (2002) observed three basic errors in numerical modeling:
• reality is replaced by mathematical theory
• mathematical theory is implemented in piecewise fashion or in differential equations for field quantities
• numerical error due to the finite precision of the computer arithmetic
• Complex models do not produce better results due to induced additional errors from the uncertainty of some parameters

There errors are to make the user aware potential warnings when utilizing numerical models. Though numerical models and LEM due contain some potential errors and uncertainty, both methods construct valuable information for rock slope analysis. Before starting any analysis, it is required the correct method that best represents the slope behavior is chosen. The next section provides a comparison between numerical modeling and the LEM.

5.2.2 Shear Strength Reduction Method

The limit equilibrium method presents a straightforward procedure to obtain the factor of safety for the critical slip surface in the rock slope; however, FLAC and UDEC utilize a more in depth method to calculate the factor of safety. Sainsbury et al (2003) stated the traditional definition of the factor of safety for slopes stability analysis is to calculate the factor of safety with respect to the soil/rock shear strength. The factor of safety of a slope can be computed with a finite element or finite difference code by reducing the rock shear strength in stages until the slope fails. The resulting factor of safety is the ratio of the actual shear strength to the reduced shear strength at failure. This method is called the shear strength reduction technique and is
described by Dawson et al (1999). The equations used for the shear strength reduction (SSR) are the following:

\[ c_{\text{trial}} = \frac{1}{F} \]  
\[ \phi_{\text{trial}} = \arctan\left\{ \frac{1}{F} \tan\phi \right\} \]

where; \(c\) is cohesion, \(\phi\) is the friction angle, and \(F\) is the factor of safety. During the reduction process, the trial parameters will keep changing until the factor of safety satisfies equations 3 and 4.

5.3 Past Comparisons of Limit Equilibrium Method & Numerical Modeling

The LEM can be considered the backbone for rock slope stability analyses. As models and demand for accuracy is becoming essential to slope stability analyses, more complex models with detailed geometry will most likely be needed; hence understanding if the LEM or numerical modeling methods would be more ideal when analyzing a rock slope. Some general comparisons between the LEM and numerical modeling are displayed in table 5.2.

Table 5.2: Comparison of numerical model solutions to LEM solutions (Wyllie & Mah 2004)
These comparisons indicate numerical solutions contain more flexibility and satisfy equilibrium conditions everywhere. Kourdey et al (2001) suggests numerical models are more appropriate for a limit equilibrium analysis due to the following advantages:

- Complex phenomena can be modeled (effect of water, dynamic, discontinuity, etc.);
- Capacity to introduce constitutive models;
- Factor of safety is not constant along the slip surface;
- The factor of safety of a slope can be computed with Itasca programs by reducing the soil shear strength in stages until the slope fails, but this does not show a well-defined surface of failure;
- Numerical programs can display two-dimensional plots of sliding plains.

Hammah et al (2005), performed a comparison study between the factor of safety obtained for the LEM and SSR. In their study, they compiled a list for numerous
slope scenarios using both methods and reported the results. The study determined in order to get similar factor of safety values for the LEM and SSR, the following must be used:

- use the same E value for the materials in a multiple-material model
- assume a single valid Poisson’s ratio for the materials
- assume a dilation angle = 0, and
- use the elastic-perfectly plastic assumption for post-peak behavior.

Also, Cala et al (2003) provide various solutions to the comparison of factor of safety values for the LEM and SSR method. This is shown in figure 5.2. Overall, both factor of safety values have their advantages and flaws, but caution is advised when analyzing the slip surface factor of safety solutions. The following section will explain further in detail a factor of safety comparison between LEM and SSR.

![Figure 5.2: Factor of safety comparison between the LEM (FS) and the SSR (FS1), Cala et al (2003)](image)
The following chapter of the dissertation will further investigate the difference in factor of safety values between the LEM and the SSR. The factor of safety is determined for rock slope at 30 m and 915 m using different rock types and wet/dry conditions.

5.4 Limit Equilibrium Method vs. Discrete Element Method

The LEM does calculate adequate solutions in equilibrium conditions; however Li et al (2007) revealed that joint structures in a rock slope have an important impact on a jointed rock slope. Their study also indicated the DEM (UDEC) simulations were in reasonable agreement with the failure modes produced by the jointed rock slope containing different joint structures. UDEC can simulate numerous failure modes which is the case in a jointed rock slope where LEM can only analyze one failure mode. Figure 5.3 summarizes the attributes contained in the LEM and UDEC.
5.5 Limit Equilibrium Method vs. Finite Difference Method

Sainsbury et al (2003) presented advantages of a finite difference solution over a limit equilibrium solution which include the following:
• Any failure mode develops naturally; there is no need to specify a pre-defined failure surface.
• No artificial parameters, such as inter-slice force angles, need to be given as input.
• Multiple failure surface or complex internal yielding evolves naturally.
• Structural interaction (rock bolts) is modeled realistically as fully coupled deforming elements, not simply as equivalent forces.
• The solution consists of mechanisms that are feasible kinematically. Limit equilibrium methods only consider forces, not kinematics.

Cala & Flisiak (2001) investigated a FLAC and LEM factor of safety and slip surface comparison for a lignite open mine in Poland and produced some interesting results. LEM analysis showed that minimum FS was equal to 2.115 (Morgenstern-Price method). SSR analysis showed considerably (80%) lower FS (1.18). The location of identified slip surface was completely different than that obtained from LEM. All critical slip surfaces identified with several LEM were located on the left side of the slope on its upper part. SSR identified critical slip surface located on the right side of the slope on its lower part. FLAC found its critical slip surface at the sharpest sloping angle due to induced stress caused by the upper section of the slope adding more driving force to the toe region of the slope. This is presented in figure 5.4 below.
Figure 5.4: Factor of safety and slip surfaces for 170 m soft rock slope (Cala & Flisiak, 2001)

It has to be taken under consideration that FS is a function of mechanical properties of rock or soil (Cala & Flisiak 2001). The input data is the most significant factor affecting the reliability of the FS; hence, rock characterization plays a critical role. Jing (2003) lists some of the rock characterization problems currently experienced:

- the in situ rock stress is not easy to characterize over the region to be modeled;
- rock properties measured in the laboratory may not represent the values on a larger scale;
- rock properties cannot be measured directly on a large scale;
- rock properties may have to be estimated from empirical characterization techniques;
- the uncertainty in the rock property estimates is not easy to quantify.
If simple geometry is used for slope stability analysis, Slope/W (LEM) and FLAC (SSR method) calculate very similar factors of safety. Again, Cala & Flisiak (2001) produced results indicating as the geometry and geology get more complex and the scale of the rock slope is large, the factor of safety values begin to differ; however, at small scale with several geology components the FS values were the same. From experience and past investigations, the SSR method has several advantages over the LEM. These advantages were presented in Cal & Flisiak (2004) and Sansbury et al (2003) and are the following:

- SSR does not need to assume failure shape and location
- Stress and strain field determine failure surface
- SSR seems to perform better over the LEM when dealing with complex geometry and geology
- SSR in finite difference codes satisfy translational and rotational equilibrium

From the provide research information available and lists of comparison between SSR vs. LEM, the user will need to make the final determination of which analyses method provides the most realistic results. For further SSR vs. LEM comparison information, refer to Hammah et al (2005).
5.6 **Continuum Modeling vs. Discontinuum modeling**

As continuum and discontinuum modeling becomes more available and common in practice, the user needs to be aware of the advantages and limitation of each type of model to ensure the most applicable scenario in a computer model. FLAC and UDEC (Itasca, 2006) are the most prevalent programs available within the mining and civil engineering industries. Table 5.3 offers the general limitations and advantages contains in discontinuum and continuum methods.


<table>
<thead>
<tr>
<th>Analysis Method</th>
<th>Critical Input Parameters</th>
<th>Advantages</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuum Modelling (e.g. finite-element, finite-difference)</td>
<td>Representative slope geometry; constitutive criteria (e.g. elastic, elasto-plastic, creep, etc); groundwater characteristics; shear strength of surfaces; in situ stress state</td>
<td>Allows for material deformation and failure. Can model complex behavior and mechanisms. Capability of 3-D modelling. Can model effects of groundwater and pore pressure. Able to assess effects of parameter variations on instability. Recent advances in computing hardware allow complex models to be solved on PC's with reasonable run times. Can incorporate creep deformation and dynamic analysis.</td>
<td>Users must be well trained, experienced and observe good modelling practice. Need to be aware of model/software limitations (e.g boundary effects, mesh aspect ratios, symmetry, hardware memory restrictions). Availability of input data generally poor. Required input parameters not routinely measured. Inability to model effects of highly jointed rock. Can be difficult to perform sensitivity analysis due to run time constraints.</td>
</tr>
<tr>
<td>Discontinuum Modelling (e.g. distinct-element, discrete-element)</td>
<td>Representative slope and discontinuity geometry; intact constitutive criteria; discontinuity stiffness and shear strength; groundwater characteristics; in situ stress state.</td>
<td>Allows for block deformation and movement of blocks relative to each other. Can model complex behavior and mechanisms (combined material and discontinuity behavior coupled with hydro-mechanical and dynamic analysis). Able to assess effects of parameter variations on instability.</td>
<td>As above, experienced user required to observe good modelling practice. General limitations similar to those listed above. Need to be aware of scale effects. Need to simulate representative discontinuity geometry (spacing, persistence, etc). Limited data on joint properties available.</td>
</tr>
</tbody>
</table>

As stated from Stead, et al. (2001), continuum modeling is best suited for the analysis of slopes that are comprised of massive, intact rock, weak rocks, and soil-like or heavily fractured rock masses; however, the continuum model is not appropriate for blocky medium. FLAC also is efficient at large strains and distortions. Sjoberg (1999) also cautioned that continuum models may not have a distinct failure surface.
and an assumption of continuum may not be realistic for large slope deformations. Kourdey et. al (2001) acknowledge how the Lagrangian calculations operated in FLAC are also well suited to modeling large distortions and allows the user to examine the development of material failure at each node. However, Zhang & Sanderson (2002) emphasized there are difficulties with the implementation of the FD techniques for problems with complex geometrical shapes and with some types of boundary conditions. Also, Jing (2003) commented the FDM cannot simulate infinitely large domains and the efficiency for the FDM will decrease at high number of degrees of freedom which relate to the number of nodes. The continuum approach can be used when blocks are interconnected and no fractures present in the analysis.

The discrete approach is most suitable for moderately fractured rock masses where fractures and discontinuities are too difficult to model in a continuum model and displacements of individual blocks is wanted. Jing (2003) suggested that large displacements caused by rigid body motion of individual blocks, including block rotation, fracture opening and complete detachments is straightforward in the DEM, but impossible in the FDM. Stead et al (2006) also stated it must be recognized that conventional discontinuum models also have inherent limitations. Failure is frequently followed or preceded by creep, progressive deformation (fatigue damage processes), and extensive internal disruption of the slope mass (brittle/ plastic damage). Staub et al. (2002) verified that while it is possible to contain joints in UDEC that terminate in intact rock, it proved difficult to maintain numerical stability in the post-peak region at large strains. Sjoberg (1999) indicated the locations of the pre-existing discontinuities need to be known before analysis takes place.
FLAC utilizes a finite difference mesh constructed on quadrilateral elements where the stress and strain is calculated for each element which can be advantageous due to the allowance of some elements containing large deformation and others available for further calculations. This was indicated by Sjoberg (1999) as well as the following mesh disadvantages for FLAC:

- Geometry can be difficult to construct due to irregular boundaries
- Elements may yield too early and overestimate the extents of failure
- Usually not feasible when modeling the hanging wall and footwall of slopes
- Can be complicated to represent localization of deformation in slope

Though the FLAC mesh generation can cause some complications with the model, this type of mesh formulation is required when dealing with large strains and high scale rock slope in a continuum model. However, UDEC does not utilize the same mesh available in FLAC. UDEC contains individual finite difference triangles which in turn can be more easily accessible than the FLAC grid generation. Again, Sjoberg (1999) stated UDEC contained the following disadvantages:

- The collapse load can be overestimated
- Not suited for models with substantial amount of yielding blocks
- Large scale models will most likely produce less accurate results for failure surface
The SSR method is a very useful method to determine the factor of safety using FLAC and UDEC; however, the SSR method has some limitations. Cala et al (2003) listed the following limitations:

- The SSR method requires advanced numerical modeling skills
- Complicated models may require several hours of calculations
- SSR only identifies one failure surface (with same FS value)

Overall, the choice of continuum or discrete methods depends on many problem-specific factors, but mainly on the problem scale and fracture system geometry Jing (2003). If time permits, UDEC and FLAC can both be used in rock slope design to cross check the slope failure surface and factor of safety. For further information and details pertaining to discontinuum and continuum modeling, refer to Chiwaye (2010), Sjoberg (1999) or Jing (2003).

5.7 Future Research

Itasca and Geo-slope International have provided some very useful computer modeling programs that have benefited the rock engineering community. Slope/W, UDEC and FLAC all have their strengths and disadvantages and new programs and research are becoming available to engineers. Within the past decade, Rockfield of UK has developed a finite element and discrete element system called ELFEN that will promote new procedures of analyzing rock slope. Eberhardt et al (2004) stated ELFEN is controlled according to a fracturing criterion specified through a constitutive model (e.g. Rankine tension, rotating crack, Mohr–Coulomb, etc.). At
some point in the analysis the adopted constitutive model predicts the formation of a
failure band within a single element or between elements. The load carrying capacity
across such localized bands decreases to zero as damage increases until eventually the
energy needed to form a discrete fracture is released. Rockfield (1999) suggested
ELFEN can analyze the following continuous and discontinuous engineering
applications:

- Linear static
- Linear & non-linear buckling
- Natural frequency
- Thermo-mechanically coupled
- Transient & transient dynamic
- Transient heat transfer
- Non-linear materials
- Geometric non-linear
- Two phase
- Non-linear heat transfer
- Steady state heat transfer
- Linear & non-linear field
- Contact and impact
- Fragmentation
- Newtonian & non-Newtonian Fluids
Rockfield (1999) also indicated ELFEN is capable of handling the following key features of the analysis modules are:

- Powerful contact procedures for representing impact and sliding between bodies including discrete element procedures
- Advanced process control procedures to enable multi-stage applications to be analysed
- Accurate material models using the most up-to-date algorithms
- Consistent finite strain plasticity/visco-plasticity algorithms
- Thermo-mechanical coupling procedures
- Automatic load control
- Element activation and de-activation
- Discrete element representation for multi-body contact applications
- Interfaces for user defined material, fracture, contact and loading models
- Automatic adaptive remeshing

At this time the author has not had the opportunity to use ELFEN for rock mechanic and slope stability analysis, but believes the program is a step towards furthering technology and knowledge in the numerical modelling field of rock mechanics and slope stability. Further information and ELFEN examples are displayed by Stead et al (2006).

Another modeling program that has caught the interest in the slope stability is the Particle Flow Code by Itasca (2004). This numerical modeling program simulates movement similar to the DEM used in UDEC and also has the capabilities to analyze rock slopes in 2D and 3D. As summarized in Wang et al (2003), PFC2D simulates the
mechanical behavior (both static and dynamic) of a material by representing it as an
assembly of circular particles that can be bonded to one another. Within the “particle
mesh” in PFC, an unlimited number of joint planes defined as sets or individually. An
example of a rock slope in PFC2D is exhibited in figure 5.5.

![Figure 5.5: Slope failure simulated in PFC2D (Wang et al 2003)](image)

Lastly, a numerical modeling program has been fully developed in past decade
called Virtual Geoscience Simulation Tool (VGeST) which began as a FEM/DEM in
1989 by Antonio Manjeza and was soon published and further constructed with
several individuals at Imperial College London until the program became VGeST in
2009. This program has shown its abilities for modeling discontinuous systems for
the field of geosciences and may eventually prove to be very beneficial in the field of
slope stability. VGeST is discontinuous modeling system that incorporates particle
flow for internal deformations in corresponding elements.
5.7 Conclusion

The purpose of this chapter was not to point out the disadvantages of the FLAC, UDEC and SLOPE/W, but to rather display the limitations, advantages and uses of stability analysis methods available to engineers analyzing and designing large rock slopes in practice. After reviewing the principle aspects and the knowledge obtained from present research, the DEM, FDM and LEM all have their purpose in analyzing large rock slopes; however, the user must understand the failure mechanisms involved in the rock slope. The LEM provides a quick estimation for rock slope factor of safety and determining the critical slip surface for slopes at static equilibrium. Though it has its limitations, the LEM has provided generous assistance to ensure mine safety and maximum ore retrieval since the production in mining has become an important aspect for living conditions. For a more detailed analysis, the engineer can utilize numerical model which are broken into discontinuum and continuum problems. Depending on heterogeneity of the rock slope, either the discontinuum or continuum model should suffice; however, if the rock slope contains complex geology at a large scale where large strain conditions are possible, FLAC should be considered over UDEC. UDEC does perform excellently with small scale jointed media.

Overall, as noted in Stead et al (2006) recommends combing the use of both the LEM and numerical modeling techniques to ensure maximum certainty and utilize the advantages to both methods. Even before building reliability in model results, the input parameters from laboratory tests and in situ data should be confirmed with extra caution. Before model constructing takes on the new role of another dimension or more complex mathematics, focus needs to be place on how accurately in situ data
can be obtained and entered into numerical models. As technology advances, our knowledge of the rock profile within the slope will enhance and modeling software results will begin to converge with more accuracy to the true value of stress, strain and the factor of safety. A further study performed by the author, is presented in Chapter 6.
CHAPTER 6: FACTOR OF SAFETY FOR ROCK SLOPES

6.1 Introduction

Numerical programs are available to assist geotechnical engineers in several tasks such as estimating the factor of safety (FOS) for the stability of the road cut or mine slopes, as well as predicting displacements and stresses. Care should be taken however as these programs will all compute different results and usually produce varying individual slip surfaces. The factor of safety values obtained from these programs must be carefully analyzed with the realization that the limit equilibrium method (LEM) and the shear strength reduction technique (SSR) may produce FOS values different from one another. As noted, Stead et al (2006) recommends combining the use of both the LEM and numerical modeling techniques to ensure maximum certainty and utilize the advantages to both methods.

Previous studies have presented comparisons of FOS values for small-scale soil or rock slopes, notable examples are found in Hammah et al. (2005), Cala & Flisiak (2001) and Cala & Flisiak (2004). These papers show that the factor of safety values computed for both LEM and SSR are reasonable and generally agree. Though these findings have been very useful for smaller scale civil engineering work, more research is needed on the applicability of both methods for large scale rock slopes with heights over 600 meters, typical for rock slopes in mining operations.

The following chapter will investigate the FOS values obtained through the use of LEM and SSR techniques for large scale rock slopes. Both homogeneous and heterogeneous materials are considered as well as the influence of in situ horizontal stress and water. This study shows how the FOS values compare when using a stress
based numerical model (SSR) technique to a moment/force based method (LEM) technique.

6.2 Methodology

Various types of scenarios were constructed using two modeling programs containing the LEM or SSR method. Three models were built for this study. The first was a simple one material, 10 meter high soil slope. This was constructed to verify the results of previous studies. The second model was a single material, 915 meter high rock slope. The last model was a multiple material 915 meter high rock slope having at least three materials and at most five materials. For each model the horizontal to vertical stress ratio ($k_o$) varied from 0.33 to 1.0 and contained both dry and wet scenarios. An overall slope angle of 45 degrees (1:1 ratio) was used for all models.

The friction angle and cohesion pairs which are derived for each LEM scenario were initially estimated using the rock engineering charts shown in Hoek & Bray (1981). Wet condition number four was utilized to determine the phreatic surface for wet conditions.

Minor adjustment of the cohesive strength was performed until the LEM produced FOS values of exactly 1.2, 1.1, and 1.0 for each scenario. These resulting strength values (friction angle and cohesion) were then input into separate numerical models for calculation of the SSR method FOS value.
This investigation computed LEM FOS values using Geostudios Slope/W and SSR technique FOS values using Itasca’s Fast Lagrangian Analysis of Continua (FLAC). A Mohr-Coulomb perfectly plastic yield criteria was assumed.

6.2.1 Limit Equilibrium Method (LEM)

The Limit Equilibrium Method (LEM) is based on static force and moment equilibrium. The method has been in widespread use since the early 20th century and may be considered the oldest method used for stability calculations in geological engineering. During this investigation the Spencer Method, a method of slices that satisfies both force and moment equilibrium, will be used.

The factor of safety is calculated as a ratio of resisting forces to driving forces and expressed in the following format:

\[
FOS = \frac{\text{Resisting Forces}}{\text{Driving Forces}} \quad (1)
\]

\[
FOS = \frac{cA + W\cos\psi \tan \phi}{W \sin \phi} \quad (2)
\]

Where; \( c \) is cohesion, \( \psi \) is the angle of the slope, \( \phi \) is the friction angle of the rock material and \( W \) is the weight of the rock mass in question. When the cohesion of the material is not present in equation (2) and the slope is dry, the rock block will slide when the dip angle of the sliding surface equals the friction angle of the rock. Stability is independent of any deformation or the size of the sliding block, and the
driving forces equal the resisting forces indicating a FOS of 1.0. A FOS of 1.0 is assumed to be the “threshold” where the rock slope goes from stable to unstable. When the FOS is less than 1.0, the driving forces overcome the resisting forces causing slope movement.

6.2.2 Shear Strength Reduction Technique (SSR)

The limit equilibrium method presents a conventional procedure to obtain the factor of safety for the critical slip surface in the rock slope; however, FLAC utilizes an alternative method to calculate the factor of safety. The traditional definition of the factor of safety for slopes stability analysis is to calculate the factor of safety with respect to the soil/rock shear strength. The factor of safety of a slope can be computed with a finite element or finite difference code by reducing the estimated rock shear strength in stages until the slope fails. The resulting factor of safety is the ratio of the estimated shear strength to the reduced shear strength at failure. This method is called the shear strength reduction technique and is described by Dawson et al (1999) [11]. The equations used for the shear strength reduction (SSR) are the following:

\[ c_{\text{trial}} = \frac{1}{F} \]  \hspace{1cm} (3)
\[ \varphi_{\text{trial}} = \arctan\left\{\frac{1}{F} \tan \varphi\right\} \]  \hspace{1cm} (4)

where; \( c \) is cohesion, \( \varphi \) is the friction angle, and \( F \) is the factor of safety. During the reduction process, the trial parameters will keep changing until the factor of safety satisfies equations 3 and 4. The FLAC program may err when the FOS falls well
below 1.0, failing to calculate a FOS. When this occurred during a simulation the FOS was reported to be less than 0.95, indicating failure.

6.3 Homogeneous Soil Slope

A model of a 10 meter high slope consisting of a single material with a unit weight of 19.6 kN/m³ was constructed. Figure 6.1 shows the LEM model that was created and Figure 6.2 displays the SSR model. A typical slip surface obtained for this model is illustrated for reference.

Figure 6.1: 10 meter high soil slope, FOS = 1.0 (LEM)
6.3.1 Material Properties

Table 6.1 reports the material properties derived for the 10 meter homogeneous soil slope model to obtain FOS values of 1.2, 1.1, and 1.0 for the LEM. These material properties were then input into a FLAC model which exactly matched the Slope/W model.
Table 6.1: Derived Material Strengths for 10 Meter Slope

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry Condition</th>
<th>1/2 Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Phi (deg)</td>
<td>Coh (kPa)</td>
</tr>
<tr>
<td>1.3</td>
<td>21</td>
<td>19.6</td>
</tr>
<tr>
<td>1.2</td>
<td>19</td>
<td>18.6</td>
</tr>
<tr>
<td>1.1</td>
<td>17</td>
<td>17.5</td>
</tr>
<tr>
<td>1.0</td>
<td>15</td>
<td>16.4</td>
</tr>
</tbody>
</table>

6.3.2 Factor of Safety Results

Table 6.2 compares the factor of safety values returned for the 10 meter homogeneous soil slope.

Table 6.2: FOS Comparison for 10 Meter Slope

\[ k_o = 1 \]

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry</th>
<th>1/2 Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEM</td>
<td>SSR</td>
</tr>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.29</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.19</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>1.09</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>
k_0 = 0.33

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry LEM</th>
<th>Dry SSR</th>
<th>1/2 Saturated LEM</th>
<th>1/2 Saturated SSR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.29</td>
<td>1.30</td>
<td>1.27</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.19</td>
<td>1.20</td>
<td>1.19</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>1.09</td>
<td>1.10</td>
<td>1.09</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.99</td>
</tr>
</tbody>
</table>

6.4 Homogeneous Rock Slope

A model of a 915 meter high slope consisting of a single material with a unit weight of 25.1 kN/m³ was constructed. Figure 6.3 shows the LEM model that was created and Figure 6.4 displays the SSR model. For illustration purposes, the slip surface obtained for a FOS value of 1.0 is shown.

Figure 6.3: 915 meter high rock slope, FOS = 1.0
6.4.1 Material Properties

Table 6.3 reports the material properties derived for the 915 meter homogeneous rock slope model to obtain FOS values of 1.2, 1.1, and 1.0 for the LEM.
Table 6.3: Derived Material Strengths for 915 Meter High Slope

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry Condition</th>
<th>1/2 Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Phi (deg)</td>
<td>Coh (kPa)</td>
</tr>
<tr>
<td>1.3</td>
<td>39</td>
<td>687</td>
</tr>
<tr>
<td>1.2</td>
<td>37</td>
<td>632.0</td>
</tr>
<tr>
<td>1.1</td>
<td>36</td>
<td>478.8</td>
</tr>
<tr>
<td>1.0</td>
<td>35</td>
<td>344.7</td>
</tr>
</tbody>
</table>

6.4.2 Factor of Safety Results

Table 6.4 compares the factor of safety values returned for the 915 meter homogeneous rock slope.

Table 6.4: FOS Comparison for 915 Meter Slope

\[ k_o = 1 \]

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry</th>
<th>1/2 Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEM</td>
<td>SSR</td>
</tr>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.18</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.09</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>0.97</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>&lt;0.95</td>
</tr>
</tbody>
</table>
\( k_0 = 0.33 \)

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry LEM</th>
<th>Dry SSR</th>
<th>1/2 Saturated LEM</th>
<th>1/2 Saturated SSR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.22</td>
<td>1.30</td>
<td>1.22</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.12</td>
<td>1.20</td>
<td>1.10</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>1.01</td>
<td>1.10</td>
<td>1.03</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>&lt;0.95</td>
<td>1.00</td>
<td>0.99</td>
</tr>
</tbody>
</table>

### 6.5 Heterogeneous Rock Slope

Real world modelling necessitates the use of multiple materials in numerical models. The heterogeneous rock slope example investigates the effect that multiple materials have on FOS estimations. All of the heterogeneous rock slope examples were built with a 915 meter high slope model.

#### 6.5.1 Three Material Model

A three material model was constructed with three different rock layers with equal heights. Figure 6.5 shows the Slope/W model that was created. For illustration purposes, the slip surface obtained for a FOS value of 1.0 is shown.
6.5.1.1 Material Properties

Table 6.5 reports the material properties derived for the 915 meter, three material type rock slope model, to obtain FOS values of 1.2, 1.1, and 1.0 for the LEM. Layer numbers start from the bottom.

Table 6.5: Derived Material Strengths for Three Material Model

<table>
<thead>
<tr>
<th>Dry Condition</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Properties</td>
<td>1.0</td>
</tr>
<tr>
<td>Layer 1 Phi (deg)</td>
<td>34</td>
</tr>
<tr>
<td>Layer 1 C (kPa)</td>
<td>359.1</td>
</tr>
<tr>
<td>Layer 2 Phi (deg)</td>
<td>35</td>
</tr>
<tr>
<td>Layer 2 C (kPa)</td>
<td>383</td>
</tr>
<tr>
<td>Layer 3 Phi (deg)</td>
<td>36</td>
</tr>
<tr>
<td>Layer 3 C (kPa)</td>
<td>397.4</td>
</tr>
</tbody>
</table>
### 6.5.1.2 Factor of Safety Results

Table 6.6 compares the factor of safety values returned for the three material rock slope model.

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>1/2 Saturated Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>Layer 1 Phi (deg)</td>
<td>45</td>
</tr>
<tr>
<td>Layer 1 C (kPa)</td>
<td>574.6</td>
</tr>
<tr>
<td>Layer 2 Phi (deg)</td>
<td>46</td>
</tr>
<tr>
<td>Layer 2 C (kPa)</td>
<td>588.9</td>
</tr>
<tr>
<td>Layer 3 Phi (deg)</td>
<td>47</td>
</tr>
<tr>
<td>Layer 3 C (kPa)</td>
<td>598.5</td>
</tr>
</tbody>
</table>

Table 6.6: FOS Comparison for Three Material Model

$k_o = 1$

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry LEM</th>
<th>Dry SSR</th>
<th>1/2 Saturated LEM</th>
<th>1/2 Saturated SSR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.12</td>
<td>1.30</td>
<td>1.27</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.06</td>
<td>1.20</td>
<td>1.01</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>0.96</td>
<td>1.10</td>
<td>1.00</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>&lt;0.95</td>
<td>1.00</td>
<td>&lt;0.95</td>
</tr>
</tbody>
</table>
\[ k_0 = 0.33 \]

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry LEM</th>
<th>Dry SSR</th>
<th>1/2 Saturated LEM</th>
<th>1/2 Saturated SSR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.17</td>
<td>1.30</td>
<td>1.26</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.08</td>
<td>1.20</td>
<td>1.03</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>0.97</td>
<td>1.10</td>
<td>0.99</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>&lt;0.95</td>
<td>1.00</td>
<td>&lt;0.95</td>
</tr>
</tbody>
</table>

6.5.2 Five Material Model

A five material model was constructed with five different rock layers with equal heights. Figure 6.6 shows the LEM model that was created and Figure 6.7 displays the SSR model. For illustration purposes, the slip surface obtained for a FOS value of 1.0 is shown.

Figure 6.6: Five rock material model, FOS = 1.0
Figure 6.7: 915 m high, dry five rock type material model, FOS = 1.2 (SSR)

6.5.2.1 Material Properties

Table 6.7 reports the material properties derived for the 915 meter, four material type rock slope model, to obtain FOS values of 1.2, 1.1, and 1.0 for the LEM. Layer numbers start from the bottom.
Table 6.7: Derived Material Strengths for Five Material Model

<table>
<thead>
<tr>
<th>Dry Condition</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Properties</td>
<td>1.0</td>
</tr>
<tr>
<td>Layer 1 Phi (deg)</td>
<td>34</td>
</tr>
<tr>
<td>Layer 1 C (kPa)</td>
<td>320.8</td>
</tr>
<tr>
<td>Layer 2 Phi (deg)</td>
<td>35</td>
</tr>
<tr>
<td>Layer 2 C (kPa)</td>
<td>325.6</td>
</tr>
<tr>
<td>Layer 3 Phi (deg)</td>
<td>36</td>
</tr>
<tr>
<td>Layer 3 C (kPa)</td>
<td>335.2</td>
</tr>
<tr>
<td>Layer 4 Phi (deg)</td>
<td>37</td>
</tr>
<tr>
<td>Layer 4 C (kPa)</td>
<td>339.9</td>
</tr>
<tr>
<td>Layer 5 Phi (deg)</td>
<td>39</td>
</tr>
<tr>
<td>Layer 5 C (kPa)</td>
<td>359.1</td>
</tr>
</tbody>
</table>
6.5.2.2  Factor of Safety Results

Table 6.8 compares the factor of safety values returned for the five material rock slope model.
Table 6.8: FOS Comparison for Five Material Model

\(k_o = 1\)

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry</th>
<th>1/2 Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEM</td>
<td>SSR</td>
</tr>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.12</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.03</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>0.97</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>&lt;0.95</td>
</tr>
</tbody>
</table>

\(k_o = 0.33\)

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry</th>
<th>1/2 Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEM</td>
<td>SSR</td>
</tr>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.16</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.08</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>0.97</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>&lt;0.95</td>
</tr>
</tbody>
</table>

6.6 Discussion

Part of the aim of this study was to verify the results of previous studies which showed that for small scale slopes, the LEM and SSR technique computes FOS values which are reasonable and in agreement.
Table 6.9: FOS Comparison for 10 Meter Slope

\( k_o = 1 \)

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry</th>
<th>1/2 Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEM</td>
<td>SSR</td>
</tr>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.29</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.19</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>1.09</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

\( k_o = 0.33 \)

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry</th>
<th>1/2 Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEM</td>
<td>SSR</td>
</tr>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.29</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.19</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>1.09</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The results of the 10 meter slope simulations show that for varying values of pore pressure and horizontal stress, the FOS value returned using the both the LEM and the SSR technique agree.

For slope heights that are typical in the mining industry, factor of safety values returned for the LEM and SSR technique diverge. When a 915 meter high rock slope was simulated, the SSR technique produced lower values than the LEM.
Table 6.10: FOS Comparison for 915 Meter Slope

\[ k_o = 1 \]

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry</th>
<th>1/2 Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEM</td>
<td>SSR</td>
</tr>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.18</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.09</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>0.97</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>&lt;0.95</td>
</tr>
</tbody>
</table>

\[ k_o = 0.33 \]

<table>
<thead>
<tr>
<th>FOS</th>
<th>Dry</th>
<th>1/2 Saturated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEM</td>
<td>SSR</td>
</tr>
<tr>
<td>1.3</td>
<td>1.30</td>
<td>1.22</td>
</tr>
<tr>
<td>1.2</td>
<td>1.20</td>
<td>1.12</td>
</tr>
<tr>
<td>1.1</td>
<td>1.10</td>
<td>1.01</td>
</tr>
<tr>
<td>1.0</td>
<td>1.00</td>
<td>&lt;0.95</td>
</tr>
</tbody>
</table>

The SSR method always produced lower values. In some cases, the SSR technique returns FOS values that are close to the LEM, and other cases return FOS values which are as much as 0.1 lower.

When heterogeneity is applied to the slope materials, the SSR technique returns more divergent results. As more materials are added to the model, the FOS
decreases. The results also diverge as the model assumptions change, either with the horizontal stress, or with the addition of pore pressure.

These differences in FOS can be attributed directly to the calculation methods. The FOS in LEM is simply the ratio of the resisting forces to driving forces. For the SSR technique, the FOS is the ratio of the strength at failure versus the initial estimated strength. At low magnitude normal stresses, as in the case of the 10 meter slope model, the two methods produce similar results. As the slope heights increase, normal stresses increase in the FLAC model and the results diverge.

This can be graphically exhibited in a normal-shear stress plot. The normal and shear stress obtained directly from a particular FLAC model zone is plotted versus the calculated normal and shear stress of a matching location at the base of a slice in the LEM model. Figures 6.8 and 6.9 present an example of each case, where the slope height is 10 meters and where the slope height is 915 meters.
Figure 6.8: Stress conditions for 10 m homogeneous soil slope, FOS = 1.1

Figure 6.9: Stress conditions for 915 m homogeneous rock slope, FOS = 1.1
6.7 Conclusions

The results obtained in this chapter have shown that for small scale slopes, the LEM and SSR technique computes FOS values which are in agreement. For high slopes, the SSR method will always produce a lower FOS value than the LEM and will generally produce a FOS value that is 0.1 lower than for LEM.

In summary, caution is advised when running one of the two methods for estimating the factor of safety. Both LEM and SSR should be considered when analyzing large-scale rock slopes for mining and civil engineering projects. The following chapter will exhibit how to monitor slope stability, what procedures that can be implemented and how remote sensing data can be used for slope stability analyses.
CHAPTER 7: SLOPE MONITORING

7.1 Introduction

Slope stability is a key parameter in the design and operation of open pit mines. It has a direct impact on the safety and production costs. To increase profitability, mining operations favor steep slopes which under the influence of changing stress regimes, geological and hydrological conditions become less stable and will eventually fail. A number of monitoring instruments and analysis techniques have been developed to foretell slope movement and impeding failure. Advanced ground-based systems such as the Slope Stability Radar (SSR) and LIDAR have shown great promise for use in open pit operations. GIS-based investigation tools have been developed to create a representative geotechnical model and to carry out 3D slope stability analysis.

These systems, however, provide a limited view of the condition of the pit and its surrounding area. A more complete picture may be obtained by integrating all available multi-scale and multi-resolution data to create spatial and temporal data coverage (displayed as maps) so that inferences about the stability of the pit and the factors influencing stability can be made.

This chapter discusses the techniques for monitoring and analysis of slope stability, highlighting the advantages and disadvantages of each technique. The concept of integrating multi-scale and multi-resolution data from sources of similar and dissimilar nature are introduced in this chapter that would allow mining engineers to get a better handle on the ground condition in the pit. This chapter also is Step 4 in the rock slope stability flow chart (Figure 7.1).
7.2 Slope Analysis Techniques

The stability of slopes is affected by a number of factors such as geometry, geological structures, material anisotropy, in-situ stresses and coupled processes such
as fluid flow and seismic loading. Techniques developed for the analysis of slope stability therefore rely on geotechnical, hydrological and topographical parameters such as cohesion, angle of internal friction, moisture, density, slope angle and orientation, water table and hydraulic conductivity. These parameters are obtained from measurements in the laboratory, in the field or remote sensing data. The most widely used techniques of kinematic and GIS-based analysis are discussed.

7.2.1 GIS-Based Analysis Method

The GIS framework is used to acquire, store, display and manage spatial and temporal data sets in vector and raster formats. If needed, the available analytical tools within the framework can be expanded through additional modules or plug-ins. Alternatively a GIS system provides the opportunity to extend the interpretive resources offered by an existing geotechnical data management and analysis system (GDMAS) by introducing the increased use of the spatial and temporal context of the data. The GIS is used to create a dynamic link with the GDMAS for improved data display, retrieval and exploration (F. Breisgau 2003, Kamp, U. et al, 2008).

A GIS-based geotechnical analysis begins with the discretization of all required site characterization parameters into appropriately sized cells. This would typically involve the creation of spatial layers by converting pit topography into digital elevation models (DEM), structural, mineralization, groundwater, geomechanical data (e.g. rock quality designation, intact rock strength, rock mass rating, fracture frequency etc.) and, slope failure types (planar, wedge, toppling and rotational) into maps and tables. Next, attributes (for example strike, dip, dip-
direction associated with structural features and zone, bench, layback and, geotechnical units related to slope failures) are added to the spatial layers. At this stage it may be necessary to generate temporal maps representing topographical and hydrological variations in terms of parameters such as slope angle, direction of sloping face and water-table depth. Available models for developing spatial data coverage are based on knowledge-based or data-driven (statistical) methods. Using fuzzy logic and neural network analysis, knowledge-based modeling requires input from site engineers in the form of ranking parameters within each component layer based on their degree of influence on slope failure. Data-driven modeling uses statistical correlation of a training data set representing the locations of various types of slope failures to compute weights for each class in a layer. These weights are used to determine the probability of failure for each cell in the discretized region of interest. The final grid cells are color coded and superimposed over DEM maps to illustrate areas of high/low risk of slope failure.

A hybrid approach (M. Cai et.al. 2007) to create slope failure maps based on the safety factor consists of using the spatial layers described in the previous paragraph as input in a deterministic model of slope stability. Various limit equilibrium solutions such as those proposed by (M.Cai et.al. 2007) can be used to calculate a set of 3D safety factors iteratively. The figure below illustrates the steps required to carry out a typical slope stability analysis using the GIS-based approach.
7.3 **Slope Monitoring Techniques**

At present, a number of state-of-the-art non-invasive technologies are available to monitor hydrological and geotechnical conditions in an open pit mine. These technologies can provide broad and highly detailed ground condition intelligence at different scales, covering the entire pit (and beyond) all the way to the spacing between joints and fractures. Recent developments in 4D or time-lapse scanning techniques have made it possible to monitor changes in ground conditions as they occur.
7.3.1 Satellite and Airborne Imagery

Starting at largest scale of interest, that of the mine itself and its surrounding areas, the use of satellite-based imagery to monitor ground conditions has grown substantially over the past decade (Hong, Y. et al 2007; Schor, H.J. and Gray, D. 2007). In particular, InSAR (Interferometric Synthetic Aperture Radar) and its more accurate variants, CRInSAR (corner reflector InSAR) and PSInSAR (permanent scatterer InSAR) have provided the opportunity to detect surface deformation with millimeter accuracy at near real-time. Signals related to slope movement in open pit mines and mine dewatering have successfully been identified and reported in the literature (McHugh, E.L. and Girard, J 2007; Peng, X. et al 2007). InSAR also allows for the detailed analysis of the surface response related to aquifer use over time. The influence of structure, geology, regional groundwater flow regimes, and production volumes can also be integrated on a site specific basis. Although the vertical resolution of InSAR is few millimeters, the lateral resolution (the lateral averaging window) is of the order of meters. The lateral resolution could be improved by multiple stacking if the ground’s surface physical characteristics persist between passes of a satellite or an aircraft. This may not always be the case, as vegetation and soil moisture change due to seasonal or other variations. In addition, lateral variations in cover reflectivity are known to give phase shifts that could be interpreted as interferometric elevation differences.

Multi-spectral and hyper-spectral imaging is often used to map moisture content, vegetation type and soil composition. The Near Infrared (NIR) imagery is particularly useful as water is a strong absorber in this part of the spectrum. NIR
surveys before sunrise have also shown great potential as a mapping tool for identifying springs and seepage areas. Hyper-spectral imagery analyzes narrower bands in the electromagnetic spectrum to extract more specific soil or land cover properties such as iron oxide, organic matter, salinity and clay content. Image resolution is typically of the order of 10-30 meters and can be integrated with other data to identify seepage areas.

Airborne electromagnetic surveys map the bulk electrical conductivity of the ground to a depth of 300 ft. With proper care and calibration using field data from boreholes, an EM survey can be used to identify underground streams that recharge aquifers or the presence of water in the ground.

7.3.2 Ground-Based System

A number of ground-based systems using electromagnetic (Slope Stability Radar) and laser (LIDAR) technologies have been developed to monitor slope deformation. These instruments have higher resolution and operate at a smaller scale than the corresponding satellite or airborne systems. Standard operating procedure calls for dividing the pit wall into sections and scanning each section separately. Because these systems are installed much closer to the face (hundreds of feet compared to tens of thousands in the case of satellite imagery), scan a small surface area and have a small footprint, they can collect deformation data with a high degree of lateral resolution and vertical accuracy.

In general, the Slope Stability Radar (SSR) has a much larger footprint than a LIDAR system, making it suitable to monitor a wider section of the pit wall in a
shorter period of time. Although the accuracy of SSR is of the order of 2 or 3 millimeters, each pixel in the radar output has a resolution of tens of meters. This resolution varies depending on the system used and the distance from the face. On the other hand, LIDAR has a footprint of sub-centimeters and therefore has a much higher resolution. This means that one can map surface joints and fractures using LIDAR and perform geomechanical analysis in order to provide an estimate of rock mass properties within hours after data has been collected. The accuracy of LIDAR for surface deformation measurement is similar to that of SSR or slightly better depending on the data collection strategy.

Similarly, ground-based Infrared spectrometers have a resolution that would be about two orders of magnitude higher than the aerial borne systems (depending on the distance from the target). A spectral imager provides the flexibility to acquire images on-demand and at various scales and view angles. These systems have been used to map soil moisture content, identify crop and soil composition in agricultural studies. When used under proper conditions, they can provide useful information about ground condition and presence of water in the rock mass. The output of a hyperspectral imaging system is a stack of images over a spectral range, referred to as an image cube, where the block image consists of two spatial dimensions and a third spectral dimension represented by its wavelength as shown in figure 2. A radiant energy value is recorded for each data point (pixel) in the image for every wavelength sampled so that a spectrum is collected for each pixel in the image.
7.3.3 Conventional Methods

The long-established methods of monitoring slope stability rely on measuring changes in the ground movement and groundwater levels. Displacement in the slope is characterized by direction, magnitude, rate and the depth of failure plane. Conventional slope monitoring employs a single method or a combination of methods to monitor one or any number of the above parameters. For example, tiltmeters and inclinometers can determine the direction, rate of slope movement, depth and lateral extent of failure plane while extensometers provide a measure of displacement magnitude. Piezometers are used to monitor water levels in an incased borehole.

A relatively new approach to monitoring slope movement is Time Domain Reflectometry (TDR). It uses a grouted coaxial cable to send an electric pulse down in a borehole. When the pulse encounters a break or deformation in the cable, it is reflected and shows up as a spike in the recorded signal. Based on the arrival time and size of the spike, the relative rate and magnitude of the slope movement along with the location of the deformation zone can be determined in real-time and a high degree of accuracy. Although TDR has interesting advantages over traditional inclinometers such as cost, faster acquisition time (minutes compare to an hour), and access to results immediately after data collection, it cannot determine absolute movement and
direction. In addition, if water infiltrates the cable, it will alter its electrical properties making it difficult interpret the results.

7.4 Data Analysis

This section reviews the types of data processing and analysis commonly performed to extract features and evaluate the physical properties of materials using satellite, airborne or ground-based technologies. The emphasis is on the application of these methodologies to slope monitoring.

7.4.1 Land cover

Soil, in general, is a function of climate, vegetation, lithology, topography, excess water, time, and human activity (Bibus and Semmel, 1977). Although a universal standard for soil classification does not exist, in many applications the exact nature of the soil is not needed. Instead, physical and mechanical properties such as texture, structure, depth, moisture, brightness, cation exchange capacity, cohesion, and friction angle are used to infer the desired information. Most land cover classification models rely on this approach to determine the heterogeneity of the top soil layers.

Satellite remote sensing is a widely used technique for land cover classification. It allows observations to be conducted over large and remote areas within a shorter time and at less cost compared to traditional field methods. Optical and microwave sensors (Table 7.1) are available in different temporal, spatial and spectral resolutions. However, over the last decade, methods based on optical images
are proven to provide the most accurate results because they contain a greater number of bands to reveal distinctive characteristics of types of soil.

Table 7.1: Remote Sensing Based Products

<table>
<thead>
<tr>
<th>Types</th>
<th>Products</th>
<th>Satellites</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optical</td>
<td>Reflectance, NDVI, EVI, LST,</td>
<td>European ERS-1 and ERS-2 satellites,</td>
</tr>
<tr>
<td></td>
<td>Hyper-spectral</td>
<td>Canadian RADARSAT,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>United States LANDSAT TM,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>European ENVISAT, IKONOS,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>QUICKBIRD, TRMM, MODIS</td>
</tr>
<tr>
<td>Microwaves</td>
<td>Soil Moisture, SAR(InSAR)</td>
<td>European ERS-1 and ERS-2 satellites,</td>
</tr>
</tbody>
</table>

It is well recognized that soil signatures (a series of values in several radio bands) reflect soil properties of different kinds. Hyper-spectral remote sensing has been shown to provide a near-laboratory-quality reflectance spectrum for each single pixel, which allows one to identify soil, vegetation and rock. For example, one case study is shown in Figure 3 where several gypsiiferous soil types were identified. There are several traditional methods to extract the main features from hyper-spectral or multispectral images for classification purposes: 1) supervised methods, 2) unsupervised method. These methods are discussed in more detail below. Supervised classification is well used in classification of soil properties. It requires users to choose references for classification. The references always include 1) training images or subset region in the whole image, 2) spectral signatures of different types of soil. The approaches of classification differ according to different reference, discussed below.

For the first kind of reference, training samples are used to construct the multivariate normal (Gaussian) probability density function to update the class
statistics, hence, the results of the classification. The main feature of this method is to estimate the statistical parameters of each class, such as the covariance matrix and the a-priori probability density function, in the case of optical imagery (Melgani and Bruzzone, 2004). The results from this method depend heavily on the representatives of training samples, which makes it difficult to control the accuracy of classification. Moreover, spatial information related to geotechnical parameters such as soil cohesion is disregarded by this method, which is the key in slope stability analysis.

Figure 7.4: Classification and Mapping of Different Gypsiferous Soil Types (Goossens, R. and Van Ranst, E 1998)
The spectral signal (signature) is considered as a series of values, for example, reflectance, for each pixel in the multi-spectral or hyper-spectral images. Each class has their own spectral signature since soils usually have huge difference in some certain bands. Classification is made by comparing spectrum of each image with spectrum from a database or laboratory experiments, in the process of Spectral Mapping approach. The Spectral mapping approach includes Spectral Angle Mapping, Spectral Unmixing and Mixture-Tuned Matched Filtering (Melgani, F. and Bruzzone, L 2004). This approach is relatively straightforward and reduces the tedious processing involved in classification problems. However, additional effort is required to find out the appropriate indicators capable of accurately capturing the variability in spectral shapes.

Unsupervised classification assumes the total numbers of classes are known while the statistics of each class are not informed. Then every pixel is set into different groups through statistical techniques. For example, K-Means method uses a cluster analysis, which considers each cluster as a class. The center of each class is arbitrarily located and iteratively repositions when the requirement of spectral separability is achieved. In the geotechnical side, some certain soils would be sources of slope failure, like swelling soils. The focus should be to map certain type of soil, instead of distribution of full soil types obtained from unsupervised methods. Thus unsupervised method is not appropriate for geotechnical analysis for soil.

Aside from traditional methods, geostatistical techniques can be used to enhance the spatial pattern in land covers without lose of spatial information. The total number of land covers is defined before classification. Next, the probability of
each land cover in pixels is calculated separately based on a defined indicator (Goovaerts, Jacquez and Marcus, 2005). Many indicators are used for the analysis of natural resources data. For example, soil texture triangle (Bogaert P., and D'Or, D, 2003) is often used in soil sciences. It helps scientists to assign properties such as physical and chemical parameters to different types of soils. Through kriging of that indicator, pixels are grouped into classes. Variograms in general, contain a large amount of spatial information, which allow for the downscaling and upscaling of remote sensing images as well as the possibility to integrate data from different sources (Atkinson and Lewis, 2000). It has been observed that in areas where complex geometry and geological structures are dominant, fitting variograms to theoretical models becomes problematic. Furthermore, the final land cover product does not represent the reality efficiently due to the smoothing effect which removes extreme values from the data set.

7.4.2 Soil moisture

Soil moisture plays an important role in hydrology, especially surface hydrology. It may influence surface runoff processes, hydraulic conductivity, water bearing capacity and water table depth. Soil moisture is also the key parameter in many ground water flow models (MODFLOW, USGS), when dealing with atmosphere-land interactions, and the connection between surface flow and groundwater flow (Maxwell, Chow and Kollet, 2007).

Remote sensing techniques provide soil moisture distribution over a larger area and at lower costs than by conventional methods. Despite differences between
optical and microwave sensors, soil moisture is obtained by calculating the reflectance of the recorded signal. In general, microwave sensors perform better because reflectance in the microwave range (C-band: 3.8 to 7.5 cm and L-band: 15.0 to 30.0 cm) is influenced primarily by the soil’s dielectric properties commonly used to measure soil moisture.

Reflectance in the near-infrared (NIR) and shortwave-infrared (SWIR) region of the spectrum (1.2–2.5 nm) is affected by the water content in the soil. Soil wetness index (McCabe, Wood and Gao, 2004) and the Gaussian soil moisture model (Michael, Lin and Susan, 2004) have been used to estimate the water content based on descriptors from NIR and SWIR images. However, more research is being conducted to improve the accuracy of soil moisture estimation from optical sensors. Most remote sensing instruments are capable of measuring the surface soil moisture to a depth of approximately 5 cm, effectively allowing for groundwater and runoff modeling with the aid of traditional instrument, such as piezometer, neutron logging (Lubczynski, M.W. and Gurwin, J., 2005).

7.4.3 Topography Index

In many case studies (Schulz, W.H, 2007, Gupta, R.P., et. al 2008 ), topography index, including slope angle and orientation are shown to dominate shallow landslides. Topography index is calculated by creating a Digital Elevation Model (DEM) using data obtained from instruments such as LIDAR or SAR with a lateral resolution better than 10 m×10 m, and vertical resolution from 0.1m to 1m.
LIDAR has been extensively used in landslide prediction. For example, landslide morphology and activity had been characterized by (Schulz, W.H, 2007) while a landslide susceptibility map had been created by (Gupta, R. P., et al. 2008; Meisina, C. and Scarabelli, S 2008).

Interferometric Synthetic Aperture Radar (InSAR) is often used to create a digital topographic map. While the magnitude of the backscattered radiation is used in soil moisture mapping, topography is derived from the phase of radar wave. Unlike traditional SAR scanning, InSAR combines the return phases of a reflected signal from two sensors separated by a baseline to create an interferogram. The phase difference (interferogram) for each point in the image is related to the topographic height of the terrain. The analysis software automatically generates the digital elevation maps. Figure 7.5 shows a typical ground-based radar system used for slope monitoring.
7.5 Integrated Slope Monitoring

Due to the complex interaction between soil, rock and water, the stability of slopes changes as the geological and hydrological conditions change. In many situations, mining and blasting operations may have a greater impact on slope stability than natural conditions. Better slope monitoring techniques and procedures are needed to increase safety and reduce production costs.

An integrated approach to slope monitoring whereby all available multi-scale and multi-resolution data are integrated into the analysis system will provide a more detailed geotechnical and hydrological picture of the pit and its surrounding area. In order to get a better handle on slope condition, spatial and temporal information must
be acquired on a regular basis. The conventional methods of slope monitoring as 
-described in section 3 provide data from a limited number of locations on site that is 
-not representative of the condition of the entire pit. Ground-based monitoring 
systems cover large areas of the pit and provide more detailed information about the 
condition of the slope. Regional geological and hydrological conditions can influence 
slope stability. In such cases, satellite or airborne imagery data could be used to 
understand regional trends and variations that would affect the pit condition. 
Spatially, conventional methods provide the finest-scale data while for ground-based, 
airborne and satellite systems, the scale of the collected data becomes coarser as the 
coverage area increases. To integrate the fine and coarse scale data, conventional 
techniques such as cokriging and its variants may be used. However, most multi-scale 
integration algorithms assume a linear relationship between the scales. Linearity can 
be assessed using a number of techniques such as determining the multi-fractal 
properties of the data sets. 

Often, large-scale processes create spatial trends that clutter variograms, the 
geostatistical tool that describes correlations. A simple, generally accepted model 
assumes that spatial data are composed of large scale deterministic variations (trends) 
and small scale stochastic variations. As discussed in the above paragraph, techniques 
such as cokriging and its variants offer several possibilities to model linear spatial 
trends. For non-linear and non-stationary trends, feed forward artificial neural 
networks (FFANN) utilizing a backpropagation learning algorithm appear to be the 
best tool available. In addition, an FFANN provides superior pattern recognition 
capabilities for developing the relationships between sources of dissimilar nature (for
example, hydrologic and geophysical properties). This approach does not require a behavioral model. Instead, input data and the inner structure of the ANN such as the number of neurons and hidden layers, the types of connections, and the direction of information flow are used to discover patterns in data sets that exhibit a non-stationary character. In addition, ANN can identify spatial anomalies present in the pattern at different scales describing both linear and non-linear effects.

In areas where measurements are sparse or access is limited, process-based models can be used to generate simulated data to fill in the gaps. Process-based modeling differs from geostatistical interpolation in that it requires a knowledge or understanding of the processes that produce the spatial or temporal variations. The mathematical form of a process-based model is derived from an understanding of the underlying processes rather than from statistical relationships. For example, slope movement over a time period can be modeled as a function of rock mass properties, slope height and angle, external influences and ultimate failure mechanism (Colesanti, 2006). Because this type of model is site specific, detailed knowledge of these parameters is imperative. Once a process-based model is developed and tested, it can be used to simulate missing data at different spatial scales. The proposed integrated slope monitoring system is depicted in the diagram below.
Slope Stability Through Remote Sensing Data

Slope stability for soil masses in mining operations and civil engineering projects is a critical issue during watershed and slope water recharge. During this recharge soil moisture and pore pressures may fluctuate and cause instability. In geotechnical engineering, the correlation between soil moisture and pore pressure is used to determine the potential for slope failure. In hydrology, soil moisture is the main parameter used in precipitation-runoff modeling. In a semi-arid environment, massive slopes fail due to run-offs caused by a heavy rain-fall. For example, several debris flows were reported in Sabino Canyon located outside of Tucson, AZ in 2006 after successive rainfalls (Magirl et al., 2007). Ground based measurements of soil
moisture are often confined to a limited area because the time and resources to carry out such data collection campaigns are prohibitive. Remote sensing techniques offer a practical alternative for mapping soil moisture over a large area. The Topography Wetness Index (TWI) which can be derived from remote sensing images is widely used to make inferences about the spatial distribution of soil moisture (Lin et al., 2006 and Qin et al., 2006). However, the relationship between soil moisture and TWI has yet to be studied in semi-arid environments. The following section will exhibit a procedure and example to understand the relationship between soil moisture and TWI in semi-arid environments.

7.6.1 Study Area

The study area is centered in the Walnut Gulch Experimental Watershed in Tombstone, Arizona (Walnut Gulch Watershed Brochure, Keefer et al. 2008), encompassing a semi-arid area of 150 square kilometers. The study area has an elevation from 1250 m to 1580 m and is covered mostly with shrub and grass. The predominant soil types within the watershed are sandy clay and clay loam. Experiments regarding hydrological processes, erosion, sedimentation, and remote sensing have been consistently conducted throughout the past 30 years in and around the study area. The soil moisture profiles used in this research were obtained from rain gauges (Figure 7.7) installed in the study area. Soil moisture values (% volumetric water content) from twenty-one stations are measured at different depths (<5cm, <15cm, etc.). In this study, the measurements conducted within the top 5cm
of the soil profile are used since they can be easily detected by both remote sensing and ground measurements.

![Figure 7.7: Walnut Gulch Watershed and rain gauges](image)

7.6.2 Data Processing

The NASA’s Shuttle Radar Topography Mission SRTM images were acquired for 12 consecutive days beginning February 11, 2000 (Figure 7.8). The extracted sub-image for the study area contains 480 x 480 pixels with a resolution of 30 m. The elevation values in the image range from 1159 m to 2235 m. After generating the DEM, the topography attributes such as slope angle and TWI were calculated. The TWI calculation is based on the ‘wetness’ index obtained from the TOPMODEL framework (Walter et al., 2002) and the underlying physical process. The index is determined as follows:
$TWI = \ln\left(\frac{\alpha}{\tan \beta}\right)$

Here $\alpha$ is the specific catchment area (catchment area draining across a unit width of contour), and $\tan(\beta)$ is the local slope of the terrain (Güntner et al., 2004). In order to reduce the amount of voids caused by radar beam’s shadow or low backscatter, the original TWI is aggregated for every adjacent 10 by 10 pixels, resulting in a new TWI map at a coarser resolution (1/120 of degree or approximately 1 km) as shown in Figure 7.9. The aggregation of index values is suggested by SRTM30 documentation from NASA (ftp://e0srp01u.ecs.nasa.gov) which reduces the computation time significantly during the subsequent integration process. Soil moisture data measured on July 29th, 2003 are used since there were 2 hours of rain-fall occurring on that day.

![SRTM image for the study area](Figure 7.8: SRTM image for the study area (NASA, 2000))
The physical value measured at each pixel is often assigned to the pixel’s centroid when processing remote sensing images. Thus the coordinates of locations for the soil moisture measurements do also need to be shifted to the centroid of the pixels. Following this step, the linear relationship between soil moisture and the TWI are examined by calculating the correlation coefficient. The value in this case is 0.6, which indicate a high linear relation when dealing with natural events.

7.6.3 Cokriging

The number of soil moisture data points on the ground is small compared to the large area of the Walnut Gulch watershed. In order to reduce the uncertainty in the soil moisture distribution and hence the slope stability, the soil moisture values for the entire area are estimated based on the TWI map using cokriging given the linear relationship between TWI and soil moisture. The main idea behind cokriging is to
model the variation in the variables under investigation, followed by an estimation of values for locations where no measurements are available. Figure 7.10 and 7.11 shows the results of cokriging and modeling respectively.

Figure 7.10: Variation in the variables and modeling (semi-variogram and cross-semi-variogram)
The final soil moisture map presents a large soil moisture variation (from 6 to 30). A visual inspection of the final soil moisture map and the original map reveals that pixels with high moisture content are found around the rim, at an elevation of 1700 m to 2000 m or the middle range of the elevations in the study area. The overall error of prediction is 1.2%, which is smaller than the estimate of pure passive microwave measurements obtained by examining the emissivity of soils for the thermal microwave radiation (Li and Islam, 1999). Therefore, the uncertainty of soil moisture values is reduced for our further slope stability analysis.
7.6.4 Slope Stability Analysis

Translational shallow landslides have been found after rain-fall and are considered to be controlled by shallow subsurface flow convergence, soil saturation and shear strength reduction in terms of soil moisture increasing (Veerle Vanacker, 2003). These factors are all influenced by surface topography. Hence, in the case of the study area, SINMAP (Pack and Tarboton, 1998) is utilized, since it combines the steady state hydrology assumption with the infinite slope stability model to quantify slope stability and requires the information from soil moisture and topography information. The factor of safety (FS) in SINMAP is calculated as:

\[
FS = \frac{C + C \cos \theta \left[ 1 - \min \left( \frac{R_a}{T \sin \theta}, 1 \right) \right] r \tan \phi}{\sin \theta}
\]

(2)

Where, R is a steady state recharge [m/h], C is overall cohesion for the soil, r is water to soil density ratio, T is the soil transmissivity [m²/h], \( \phi \) is the internal friction angle of the soil [°] and \( \theta \) is slope angle [°].

The soil moisture parameters \( \phi \), r, and T for the Walnut Gulch study area were obtained by data collected by other researchers. The calculation of FS are then simplified and shown in the Figure 7.12.
The most unstable areas of the watershed are located in the southeastern part of the study area. By visually examining the slope angle map, the most unstable parts also have steeper angle, ranging from 40° to 70°. Hence, it is partially indicating that the slope stability is quite influenced by the slope angle.
7.7 Conclusions

This chapter reviewed existing techniques for monitoring and analysis of slope stability. Also, this chapter proposed a system where all available data obtained at different scales and from sources of similar and dissimilar nature can be integrated into a unified system. This system is especially suited for large-scale surface mining operations as it provides the mine managers with a timely and holistic view of the condition of the pit. The advantage of this system is that once the data are analyzed and integrated into spatial and temporal data coverage (displayed as maps), inferences regarding ground water flow patterns, rock mass condition and overall stability of the pit can be made.

Soil moisture was found to correlate well with topography after rain-fall during the monsoon season in a semi-arid environment. The uncertainty in the soil moisture distribution in a large area could be significantly reduced through integration of single measurements on the ground and remote sensing data. The accuracy of DEM obtained from remote sensing images need to be improved. Remote sensing sources other than STRM (for example LIDAR), could help improve the accuracy and resolution of the DEM. Increasing the number of sites where measurements of soil moisture could be conducted will also improve the accuracy of estimations.
CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary of Findings

The primary objective for the research performed in this dissertation was to ensure and maintain rock slope stability during the design and production stages in the mining and civil engineering industries. The following are specific objectives that were addressed in the dissertation:

- Introduced a simple procedure to ensure rock slope stability.
- Investigated field methods and measurements used to construct a rock strength and structure database for slope stability analysis (Chapter 2, 3).
- Analyzed rock properties for correlations, relationships and alternative methods for RQD and rock abrasion (Chapter 3, 4).
- Researched and analyzed correct procedures for slope stability modeling using methods prevalent in practice (Chapter 5).
- Comprehended the differences in factor of safety values between computer modeling programs for rock slope stability analysis (Chapter 6).
- Constructed an integrated slope stability monitoring procedure and provide evidence of its use through remote sensing (Chapter 7).

8.2 Recommendations

The following section exhibits recommendations that can be made for each rock slope stability step featured in the flow chart provided in Figure 1.1 of this dissertation. Step 1 of the flow chart is entitled Field Investigations; however, the
author only researched one important aspect of field investigations. The following recommendations can be made for determining rock orientation:

- Check core orientations near the final pit wall to verify the surface mapping and to determine whether discontinuity orientations change with depth
- Advancements in core orientation should be considered to solidify the truth orientations of the rock core
- All the core orientation methods serve their purpose; however, choose the method that best matches the predicted rock mass encountered

Step 2 of the rock slope stability flow chart is entitled Material Testing and Rock Properties Database. Rock properties are essential to determine and model the stability of rock slopes. The author provided a testing program and illustrated how rock abrasion is an important “special” testing property that may be useful for rock properties such as shear strength, RQD, RMR and RQD. The following recommendations can be made for material testing and rock abrasion:

- Test numerous amounts of rock samples in order to produce larger ranges for RQD and LA Abrasion
- Utilize direct abrasion methods such as the RAT, concrete abrasion apparatus or gouging apparatus to obtain abrasion wear
- Perform tests on all rock types and not solely on metamorphic rock (quartzite)
- The abrasion process appears to follow erodibility (Annandale, 1995) and scour
• Samples should be packaged with extreme care and be transported in a secure location

Step 3 of the rock slope stability flow chart is entitled Slope Modeling. All methods of slope modeling are currently being administered for slope stability analyses. The author provided a testing program and illustrated how rock abrasion is an important “special” testing property that may be useful for rock properties such as shear strength, RQD, RMR and RQD. The following recommendations can be made for slope modeling:

• Always used both LEM and SSR to compare results

• Utilize 3D modeling programs or more advanced programs (ELFEN, Particle Flow, etc) to compare to 2D modeling programs

• Create a larger enough models to avoid boundary effects

• Be aware of advancements, new technology and model limitations in order to understand results computed from rock slope stability models

• Choose carefully of which characteristics of rock (discontinuities, RQD, shear strength, etc.) that dictate its behavior

• Always be aware of the path taken for the critical slip surface

• Keep models simple; however, when more materials and higher slope heights are added, the FOS will increasingly differ between each method

Finally, step 4 entitled Slope Monitoring provides awareness of unstable conditions, available techniques available and an integrated slope stability approach
can be utilized to determine the factor of safety for rock slopes. The following are recommendations when using slope monitoring systems.

- Increase the size or number of sites to improve accuracy of factor of safety estimation
- Besides soil moisture, remote sensing can be used for other pertinent variables to map factor of safety in an integrated manner
- Resolution of the DEM needs to be improved
- Slope movement should be modeled as a function of rock mass properties, slope height and angle, external influences and ultimate failure mechanisms (Colesanti, 2006)
- Use process-based models to fill gaps in data sets where measurements are limited
- Better slope monitoring techniques and procedures are needed to increase safety and reduce production costs.

8.3 **Conclusions**

In summary, this dissertation was created from a general interest and professional experience in rock slope stability and investigates key factors from the four significant steps (field, lab, modeling, monitoring) as provided in Figure 1.1 to ensure rock slope stability. The dissertation was constructed based on previously published material where the dissertation author was lead author for the published material. The publications used are mentioned in the acknowledgment. The previous
chapters provided reviews, new and presently used procedures for rock slope stability, results, recommendations and conclusions.

The findings in chapter 2 were used to promote the companies of the orientation methods and help the user determine which method would be advantageous in different mining scenarios. It was not intended to promote one specific product, but to show how each method has its benefits and limitations. Errors and uncertainties introduced by personnel that may result in low-quality image logs or unreliable discontinuity orientation data include:

- Inexperienced or untrained survey operator (OBI/ABI only).
- Improper and/or infrequent maintenance of survey probes (OBI/ABI only).
- Improper data processing and optimization.
- Logging by inexperienced or poorly trained personnel that lack knowledge of the local geology and ground conditions.
- Inconsistent means of data collection by the core logger such as orientation of undesired structures. For example, measurable open joints are the primary structures of interest for slope stability analyses. Orientation of healed fractures and/or veins may result in inaccurate characterizations of the site’s dominant rock fabric orientations.
- Careless handling of core from drill rod to sleeve.
- Inaccurate transfer of orientation from core (OBI/ABI exclusive).
Chapter 2 was constructed to help engineers, drillers and geologists better understand rock core orientation methods and how to determine which method is most appropriate for varying mining or civil engineering scenarios. The four methods discussed in this chapter have all shown great potential towards obtaining true rock fabric orientations and have assisted in countless engineering projects to identify unstable conditions.

Chapter 4 displayed predicting abrasion resistance and correlating it to other rock parameters such as RQD, shear strength, UCS or joint stiffness can be an intensively time consuming task. The purpose of chapters 3 was to introduce testing methods used to obtain rock strength properties and which tests would be utilized for data interpretation in Chapter 4. Chapter 4 used the tests described in Chapter 3 and was to inform the engineering community of potential future research for rock abrasion and suggest possible methods to obtain reliable data. The following conclusions can made from the research and literature review performed in chapter 4:

- New methods for RQD and Abrasion estimates need to be investigated and established to ensure the most accurate values
- Rock abrasion can be studied using apparatuses constructed for different materials than rock.
- Ideas are out there to potentially acquire more accurate values
- Correlations are available; however, many provide inconsistencies or are based on a small database
- Previous scour and erosion investigations and techniques can be applied to rock abrasion
- Apply testing from different fields to measure rock abrasion
• Data from more rock types should be collected
• Results should be determined from direct abrasion data
• Perform abrasion resistance testing to many more specimens rather than only rock aggregates
• Need a method that causes little disturbance to estimate RQD
• RQD is closely related to shear strength including cohesion and friction angle
• Perform periodic surveys and archive data in a easily retrievable manner
• The abrasion process can strictly governed by jacking and dislodgement, dislodgement and abrasion or abrasion only
• Should Investigate all variables effecting abrasion resistance

As technology advances and new ideas are introduced to the engineering community, rock abrasion resistance will become easier and more accurately measured for rock properties. The abrasive properties of rock will hopefully provide valuable information that can be used with future correlations and previous erodibility and rock scour studies. The purpose Chapter 5 was not to point out the disadvantages of the FLAC, UDEC and SLOPE/W, but to rather display the limitations, advantages and uses of stability analysis methods available to engineers analyzing and designing large rock slopes in practice. After reviewing the principle aspects and the knowledge obtained from present research, the DEM, FDM and LEM all have their purpose in analyzing large rock slopes; however, the user must understand the failure mechanisms involved in the rock slope. The LEM provides a quick estimation for rock slope factor of safety and determining the critical slip surface for slopes at static equilibrium. Though it has its limitations, the LEM has provided generous assistance to ensure mine safety and
maximum ore retrieval since the production in mining has become an important aspect for living conditions. For a more detailed analysis, the engineer can utilize numerical model which are broken into discontinuum and continuum problems. Depending on heterogeneity of the rock slope, either the discontinuum or continuum model should suffice; however, if the rock slope contains complex geology at a large scale where large strain conditions are possible, FLAC should be considered over UDEC. UDEC does perform excellently with small scale jointed media.

Overall, as noted in Stead et al (2006) recommends combing the use of both the LEM and numerical modeling techniques to ensure maximum certainty and utilize the advantages to both methods. Even before building reliability in model results, the input parameters from laboratory tests and in situ data should be confirmed with extra caution. Before model constructing takes on the new role of another dimension or more complex mathematics, focus needs to be place on how accurately in situ data can be obtained and entered in to numerical models. As technology advances, our knowledge of the rock profile within the slope will enhance and modeling software results will begin to converge with more accuracy to the true value of stress, strain and the factor of safety. Chapter 6 reinforces the differences in results between the LEM and SSR and also introduces results for large scale rock slopes.

The results obtained in Chapter 6 have shown that for small scale slopes, the LEM and SSR technique computes FOS values which are in agreement. For high slopes, the SSR method will always produce a lower FOS value than the LEM and will generally produce a FOS value that is 0.1 lower than for LEM. In summary, caution is advised when running one of the two methods for estimating the factor of
safety. Both LEM and SSR should be considered when analyzing large-scale rock slopes for mining and civil engineering projects. After modeling is complete and slopes are constructed to the recommended design, slope monitoring systems can be installed to ensure safety for employees and predict when slopes may become unstable.

Chapter 7 reviewed existing techniques for monitoring and analysis of slope stability. Also, this chapter proposed a system where all available data obtained at different scales and from sources of similar and dissimilar nature can be integrated into a unified system. This system is especially suited for large-scale surface mining operations as it provides the mine managers with a timely and holistic view of the condition of the pit. The advantage of this system is that once the data are analyzed and integrated into spatial and temporal data coverage (displayed as maps), inferences regarding ground water flow patterns, rock mass condition and overall stability of the pit can be made.

Soil moisture was found to correlate well with topography after rain-fall during the monsoon season in a semi-arid environment. The uncertainty in the soil moisture distribution in a large area could be significantly reduced through integration of single measurements on the ground and remote sensing data. The accuracy of DEM obtained from remote sensing images need to be improved. Remote sensing sources other than STRM (for example LIDAR), could help improve the accuracy and resolution of the DEM. Increasing the number of sites where measurements of soil moisture could be conducted will also improve the accuracy of estimations.
Even before building reliability in model results, the input parameters from laboratory tests and in situ data should be confirmed with extra caution. Karl Terzaghi once remarked, “Unfortunately, soils are made by nature and not by man, and the products of nature are always complex. As soon as we pass from steel and concrete to earth, the omnipotence of theory ceases to exist. Natural soil is never uniform. Its properties change from point to point while our knowledge of its properties are limited to those few spots at which the samples have been collected. In soil mechanics the accuracy of computed results never exceeds that of a crude estimate, and the principal function of theory consists in teaching us what and how to observe in the field.” This thought can also be applied to rock materials within a slope. Before model constructing takes on the new role of another dimension or more complex mathematics, focus needs to be placed on how accurately in situ data can be obtained and entered in to numerical models. As technology advances, our knowledge of the rock profile within the slope will enhance and modeling software results will begin to converge with more accuracy to the true value of stress, strain and the factor of safety.
APPENDIX A – ALL ROCK STRENGTH CRITERIA FROM LAB TESTING

Test Hole 1 - Intact Rock Strength Criterion

\[ y = 4.209x + 4093.5 \]
\[ R^2 = 0.6676 \]
Test Hole 2 - Intact Rock Strength Criterion

\[ y = 4.1194x + 3451 \]
\[ R^2 = 0.9618 \]

Test Hole 3 - Intact Rock Strength Criterion

\[ y = 7.7646x + 17265 \]
\[ R^2 = 0.5709 \]
Test Hole 4 - Intact Rock Strength Criterion

\[ y = 4.5948x + 11248 \]
\[ R^2 = 0.3651 \]

Test Hole 5 - Intact Rock Strength Criterion

\[ y = 5.7461x + 7474.9 \]
\[ R^2 = 0.4358 \]
Test Hole 6 - Intact Rock Strength Criterion

\[ y = 6.1608x + 7103.4 \]
\[ R^2 = 0.907 \]

Test Hole 7 - Intact Rock Strength Criterion

\[ y = 9.0185x + 7117.5 \]
\[ R^2 = 0.9057 \]
APPENDIX B – ALL LA ABRASION TESTING SHEETS

Los Angeles Abrasion
ASTM C131, AASHYO T-82

PROJECT: Crushed Rock Core Testing
PROJECT LOCATION: Tucson, AZ 85721-10012
CLIENT: U of A Mines Building
LAB SAMPLE NUMBER: 357245
SAMPLE BY: Client
SUBMITTED BY: DHM
DATE SAMPLED: 2/22/2013
DATE SUBMITTED: 2/28/2013
MATERIAL SOURCE: Crushed rock cores
MATERIAL DESCRIPTION: DH-19-431
TEST PROCEDURE: ASTM C131 Grading A
TESTED BY: JMB
DATE TESTED: 3/7/2013

Grading of Test Samples

<table>
<thead>
<tr>
<th>SIEVE SIZE</th>
<th>RETAINED</th>
<th>PASSING</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>1” - 1/2”</td>
<td>1”</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td>1250 ± 25</td>
</tr>
<tr>
<td>1”</td>
<td>1/4”</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td>1250 ± 25</td>
</tr>
<tr>
<td>3/4”</td>
<td>1/4”</td>
<td>2000 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
</tr>
<tr>
<td>3/8”</td>
<td>1/4”</td>
<td>2000 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
</tr>
<tr>
<td>1/4”</td>
<td>1/8”</td>
<td>2000 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
</tr>
<tr>
<td>#4</td>
<td>#8</td>
<td>2000 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
</tr>
<tr>
<td>TOTAL</td>
<td>#8</td>
<td>5000 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
</tr>
</tbody>
</table>

Spheres: 12

Percent Loss = (Total Weight - Weight of #12) / Total Weight

WT. OF TEST FRACTIONS

# REVOLUTIONS: 500
WT. OF #12 MATL: 3550
PERCENT LOSS: 29

TOTAL 5003
LOS ANGELES ABRASION
ASTM C131, AASHTO T-96

PROJECT: Crushed Rock Core Testing
PROJECT LOCATION: Tucson AZ 85721-10012
PROJECT NUMBER: 130301TT
LAB SAMPLE NUMBER: 387545

CLIENT: U of A Mines Building
SAMPLED BY: Client
SUBMITTED BY: DWH

MATERIAL SOURCE: LK
DATE SAMPLED: 2/22/2013
DATE SUBMITTED: 2/28/2013

MATERIAL DESCRIPTION: Crushed rock cores
SAMPLE LOCATION: DH-11-918, (DH-E8-150)
TEST PROCEDURE: ASTM C131 Grading A
TESTED BY: JMD
DATE TESTED: 3/5/2013

<table>
<thead>
<tr>
<th>SIEVE SIZE</th>
<th>RETAINED</th>
<th>PASSING</th>
<th>WEIGHT OF INDICATED SIZES, g</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1/2&quot;</td>
<td>1&quot;</td>
<td>1.250 ± 15</td>
<td></td>
</tr>
<tr>
<td>1&quot;</td>
<td>24&quot;</td>
<td>1.250 ± 15</td>
<td></td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1/2&quot;</td>
<td>2.500 ± 10</td>
<td></td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>3/8&quot;</td>
<td>2.500 ± 10</td>
<td></td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1/4&quot;</td>
<td>2.500 ± 10</td>
<td></td>
</tr>
<tr>
<td>1/4&quot;</td>
<td>#4</td>
<td>2.500 ± 10</td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td>#8</td>
<td>5.000 ± 10</td>
<td></td>
</tr>
</tbody>
</table>

TOTAL: 5000 ± 10

Percent Loss = (Total Weight - Weight of #12) / Total Weight

WT. OF TEST FRACTIONS BEFORE TEST

<table>
<thead>
<tr>
<th>FRACTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
</tr>
<tr>
<td>3/4&quot;</td>
</tr>
<tr>
<td>1/2&quot;</td>
</tr>
<tr>
<td>3/8&quot;</td>
</tr>
<tr>
<td>1/4&quot;</td>
</tr>
<tr>
<td>#4</td>
</tr>
</tbody>
</table>

TOTAL: 5009

# REVOLUTIONS: 500
WT. OF #12 MATL: 2922
PERCENT LOSS: 42
**LOS ANGELES ABRASION**

**ASTM C131, AASHTO T-26**

**PROJECT:** Crushed Rock Core Testing  
**PROJECT NUMBER:** 1303037T

**PROJECT LOCATION:** Tucson AZ 85721-10012  
**LAB SAMPLE NUMBER:** 387544

**CUBIT: U of A Mines Building**  
**SAMPLED BY:** Client  
**SUBMITTED BY:** DWH

**MATERIAL SOURCE:** U/N  
**DATE SAMPLED:** 2/22/2013  
**DATE SUBMITTED:** 2/28/2013

**MATERIAL DESCRIPTION:** Crushed rock cores  
**SAMPLE LOCATION:** DYH-25700

**TEST PROCEDURE:** ASTM C131 Grading A  
**TESTED BY:** JMB  
**DATE TESTED:** 2/27/2013

---

### Gradings of Test Samples

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Retained</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot; - 1 1/2&quot;</td>
<td>1&quot;</td>
<td>1250 ± 25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1&quot;</td>
<td>3/4&quot;</td>
<td>1250 ± 25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1/2&quot;</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>3/8&quot;</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1/4&quot;</td>
<td>1250 ± 25</td>
<td>2500 ± 10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4&quot;</td>
<td>#4</td>
<td>2500 ± 10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td>#8</td>
<td>2500 ± 10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#8</td>
<td>#16</td>
<td>2500 ± 10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td>6000 ± 10</td>
<td>6000 ± 10</td>
<td>6000 ± 10</td>
<td>6000 ± 10</td>
</tr>
</tbody>
</table>

**SPHERES**

<table>
<thead>
<tr>
<th>1&quot;</th>
<th>1 1/2&quot;</th>
<th>3/4&quot;</th>
<th>1/2&quot;</th>
<th>3/8&quot;</th>
<th>1/4&quot;</th>
<th>#4</th>
<th>#8</th>
<th>#16</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>11</td>
<td>9</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Percent Loss = (Total Weight - Weight of #12) / Total Weight**

**WT. OF TEST FRACTIONS BEFORE TEST**

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>WT. OF + #12 MATL:</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>1252</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1253</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>1250</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1253</td>
</tr>
<tr>
<td>1/4&quot;</td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td></td>
</tr>
<tr>
<td>#8</td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td>5200</td>
</tr>
</tbody>
</table>

---

**# REVOLUTIONS:** 500  
**RECORDS:** 2910  
**PERCENT LOSS:** 35
LOS ANGELES ABRASION  
ASTM C131, AASHTO T-96

PROJECT: Crushed Rock Core Testing  
PROJECT NUMBER: 130303TT
PROJECT LOCATION: Tucson AZ 85721-10012  
LAB SAMPLE NUMBER: 387543
CLIENT: U of A Mines Building  
SAMPLED BY: Client  
SUBMITTED BY: DWM
MATERIAL SOURCE: Crushed rock cores  
DATE SAMPLED: 2/22/2013  
DATE SUBMITTED: 2/25/2013
SAMPLE LOCATION: DM-E6-746, (Supplemented by DM-ET-1152)
TEST PROCEDURE: ASTM C131, Grading A  
TESTED BY: JMD  
DATE TESTED: 2/27/2013

### Grading of Test Samples

<table>
<thead>
<tr>
<th>SIEVE SIZE</th>
<th>PASSING</th>
<th>RETAINED</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - 1 1/2&quot;</td>
<td>1&quot;</td>
<td>1/2&quot;</td>
<td>3/4&quot;</td>
<td>1&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 1/4&quot;</td>
<td>3/4&quot;</td>
<td>1/2&quot;</td>
<td>3/8&quot;</td>
<td>1/4&quot;</td>
<td>1/4&quot;</td>
<td></td>
</tr>
<tr>
<td>1/4&quot;</td>
<td>1/4&quot;</td>
<td>1/4&quot;</td>
<td></td>
<td>1/4&quot;</td>
<td>1/4&quot;</td>
<td>1/4&quot;</td>
</tr>
<tr>
<td>#8</td>
<td>#8</td>
<td>#8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TOTAL** | 5000 ± 10 | 5000 ± 10 | 5000 ± 10 | 5000 ± 10 |
**SPHERES** | 12 | 11 | 8 | 6 |

**Percent Loss** = (Total Weight - Weight of #12) / Total Weight

<table>
<thead>
<tr>
<th>WT. OF TEST FRACTIONS BEFORE TEST</th>
<th># REVOLUTIONS:</th>
<th>WT. OF #12 MATL:</th>
<th>PERCENT LOSS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>1251</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1250</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>1252</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1251</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4&quot;</td>
<td>1252</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td>5004</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
LOS ANGELES ABRASION
ASTM C131, AASHTO T-96

PROJECT: Crushed Rock Core Testing
PROJECT NUMBER: 130303TT
PROJECT LOCATION: Tucson AZ 85721-10013
LAB SAMPLE NUMBER: 3871542
CLIENT: U of A Mines Building
SAMPLED BY: Client
SUBMITTED BY: DWH
MATERIAL SOURCE: 1/2" Crushed rock cores
DATE SAMPLED: 2/22/2013
DATE SUBMITTED: 2/25/2013
SAMPLE LOCATION: DH-27-746, (Supplemented by DH-85-1643)
TEST PROCEDURE: ASTM C131 Grading A
TESTED BY: JMB
DATE TESTED: 2/26/2013

<table>
<thead>
<tr>
<th>SIEVE SIZE</th>
<th>RETAINED</th>
<th>WEIGHT OF INDICATED SIZES, g</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 1/2&quot;</td>
<td>1&quot;</td>
<td>1250 ± 25</td>
</tr>
<tr>
<td>1&quot;</td>
<td>3/4&quot;</td>
<td>1250 ± 25</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1/2&quot;</td>
<td>1250 ± 25</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>3/8&quot;</td>
<td>1250 ± 25</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1/4&quot;</td>
<td>1250 ± 25</td>
</tr>
<tr>
<td>1/4&quot;</td>
<td>#4</td>
<td>1250 ± 25</td>
</tr>
<tr>
<td>#4</td>
<td>#8</td>
<td>1250 ± 25</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td>5000 ± 10</td>
</tr>
</tbody>
</table>

Percent Loss = (Total Weight - Weight of #12) / Total Weight

<table>
<thead>
<tr>
<th>WT. OF TEST FRACTIONS BEFORE TEST</th>
<th># REVOLUTIONS: 500</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>1252</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1251</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>1250</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1251</td>
</tr>
<tr>
<td>1/4&quot;</td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td></td>
</tr>
<tr>
<td>#8</td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td>5004</td>
</tr>
</tbody>
</table>

WT. OF #12 MATL: 2890
PERCENT LOSS: 42
LOS ANGELES ABRASION
ASTM C131, AASHTO T26

PROJECT: Crushed Rock Core Testing
PROJECT LOCATION: Tucson, AZ 85721-10112
LAB SAMPLE NUMBER: 387641

CLIENT: U of A Mines Building
SAMPLER: Client
SUBMITTED: DH

MATERIAL SOURCE: Crushed rock cores
SAMPLE LOCATION: C-123-436

TEST PROCEDURE: ASTM C131
TESTER: Job
DATE TESTED: 2/25/2013


grinding of test samples

<table>
<thead>
<tr>
<th>sieve size</th>
<th>weight of indicated sizes g</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 + 1/2&quot;</td>
<td>1200 x 10</td>
</tr>
<tr>
<td>1&quot;</td>
<td>1200 x 15</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1/2&quot; 1200 x 25</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>1200 x 25</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1/4&quot; 2500 x 10</td>
</tr>
<tr>
<td>#4</td>
<td>#4 2500 x 10</td>
</tr>
<tr>
<td>total</td>
<td>4000 x 15</td>
</tr>
</tbody>
</table>

SPEcies: 12 11 9 6 5

Percent Loss = (Total Weight - Weight of #12) / Total Weight

wt. of test

<table>
<thead>
<tr>
<th>fractions</th>
<th>revolutions</th>
<th>wt. of #12 matl.</th>
<th>percent loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>1228</td>
<td>1220</td>
<td>27</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1220</td>
<td>1220</td>
<td>27</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>1227</td>
<td>1227</td>
<td>27</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1251</td>
<td>1251</td>
<td>27</td>
</tr>
<tr>
<td>#4</td>
<td></td>
<td>1220</td>
<td>27</td>
</tr>
<tr>
<td>total</td>
<td>4996</td>
<td>4996</td>
<td>27</td>
</tr>
</tbody>
</table>
LOS ANGELES ABRASION
ASTM C131, AASHTO T-96

PROJECT: Crushed Rock Core Testing
PROJECT NUMBER: 130353TT
PROJECT LOCATION: Tucson AZ 85721-10012
LAB SAMPLE NUMBER: 387340
CLIENT: U of A Mines Building
SAMPLED BY: Client
SUBMITTED BY: DWH
MATERIAL SOURCE: DK
DATED SAMPLED: 2/22/2013
DATE SUBMITTED: 2/28/2013
MATERIAL DESCRIPTION: Crushed rock core
SAMPLE LOCATION: DH-19-1617
TEST PROCEDURE: ASTM C131 Grading A
TESTED BY: JMB
DATE TESTED: 2/26/2013

GRADING OF TEST SAMPLES

<table>
<thead>
<tr>
<th>SIEVE SIZE</th>
<th>PASSING</th>
<th>RETAINED</th>
<th>WEIGHT OF INDICATED SIZES, g</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - 1/2</td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>1&quot;</td>
<td>1250 ± 10</td>
<td>1200 ± 10</td>
<td>2500 ± 10</td>
</tr>
<tr>
<td>1&quot;</td>
<td>1250 ± 10</td>
<td>1200 ± 10</td>
<td>2500 ± 10</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1/2&quot;</td>
<td>1250 ± 10</td>
<td>2500 ± 10</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>3/8&quot;</td>
<td>1250 ± 10</td>
<td>2500 ± 10</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1/4&quot;</td>
<td>#4</td>
<td>2500 ± 10</td>
</tr>
<tr>
<td>#4</td>
<td>#8</td>
<td>#8</td>
<td>6000 ± 10</td>
</tr>
<tr>
<td>TOTAL</td>
<td>12</td>
<td>11</td>
<td>8</td>
</tr>
</tbody>
</table>

Percent Loss = (Total Weight - Weight of #12) / Total Weight

WT. OF TEST
FRACTIONS BEFORE TEST

<table>
<thead>
<tr>
<th>FRACTION</th>
<th>WT. OF #12 MATL.</th>
<th>PERCENT LOSS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>1253</td>
<td></td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>1251</td>
<td></td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>1249</td>
<td></td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>1263</td>
<td></td>
</tr>
<tr>
<td>1/4&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td>5906</td>
<td></td>
</tr>
</tbody>
</table>

# REVOLUTIONS: 590
## APPENDIX C: ROCK PROPERTIES DATABASE

<table>
<thead>
<tr>
<th>Drill Hole #</th>
<th>Depth (ft)</th>
<th>RQD</th>
<th>Test Type</th>
<th>Lithology</th>
<th>UCS Hardness</th>
<th>Mohs' Hardness</th>
<th>Density (pcf)</th>
<th>σ1 (psi)</th>
<th>σ3 (psi)</th>
<th>Failure Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>427.0</td>
<td>45</td>
<td>Triax</td>
<td>Hornfels</td>
<td>3</td>
<td>---</td>
<td>150.0</td>
<td>9901</td>
<td>2000</td>
<td>Fracture</td>
</tr>
<tr>
<td>1</td>
<td>688.0</td>
<td>84</td>
<td>Disk</td>
<td>Hornfels</td>
<td>3</td>
<td>---</td>
<td>158.1</td>
<td>0</td>
<td>-580</td>
<td>Both</td>
</tr>
<tr>
<td>1</td>
<td>688.0</td>
<td>84</td>
<td>Triax</td>
<td>Hornfels</td>
<td>3</td>
<td>---</td>
<td>162.0</td>
<td>8039</td>
<td>1000</td>
<td>Fracture</td>
</tr>
<tr>
<td>1</td>
<td>727.0</td>
<td>80</td>
<td>Triax</td>
<td>Hornfels</td>
<td>3</td>
<td>---</td>
<td>154.2</td>
<td>12829</td>
<td>2000</td>
<td>Fracture</td>
</tr>
<tr>
<td>2</td>
<td>918.0</td>
<td>56</td>
<td>Disk</td>
<td>Quartzite</td>
<td>2</td>
<td>---</td>
<td>140.0</td>
<td>0</td>
<td>-1021</td>
<td>Intact</td>
</tr>
<tr>
<td>2</td>
<td>918.0</td>
<td>56</td>
<td>Triax</td>
<td>Quartzite</td>
<td>2</td>
<td>---</td>
<td>149.7</td>
<td>10347</td>
<td>2000</td>
<td>Fracture</td>
</tr>
<tr>
<td>2</td>
<td>1540.0</td>
<td>87</td>
<td>Disk</td>
<td>Quartzite</td>
<td>2</td>
<td>---</td>
<td>130.8</td>
<td>11344</td>
<td>2000</td>
<td>Fracture</td>
</tr>
<tr>
<td>2</td>
<td>1540.0</td>
<td>87</td>
<td>Triax</td>
<td>Quartzite</td>
<td>2</td>
<td>---</td>
<td>132.3</td>
<td>13492</td>
<td>2000</td>
<td>Both</td>
</tr>
<tr>
<td>2</td>
<td>1671.0</td>
<td>33</td>
<td>Triax</td>
<td>Quartzite</td>
<td>2</td>
<td>---</td>
<td>161.3</td>
<td>0</td>
<td>-2381</td>
<td>Intact</td>
</tr>
<tr>
<td>3</td>
<td>3442.0</td>
<td>65</td>
<td>Disk</td>
<td>Quartzite</td>
<td>5</td>
<td>---</td>
<td>159.0</td>
<td>8409</td>
<td>0</td>
<td>Both</td>
</tr>
<tr>
<td>3</td>
<td>431.0</td>
<td>79</td>
<td>Disk</td>
<td>Quartzite</td>
<td>5</td>
<td>---</td>
<td>161.3</td>
<td>0</td>
<td>-2381</td>
<td>Intact</td>
</tr>
<tr>
<td>3</td>
<td>431.0</td>
<td>79</td>
<td>Uni</td>
<td>Quartzite</td>
<td>4</td>
<td>---</td>
<td>165.8</td>
<td>0</td>
<td>-2817</td>
<td>Intact</td>
</tr>
<tr>
<td>3</td>
<td>1786.0</td>
<td>86</td>
<td>Disk</td>
<td>Quartzite</td>
<td>5</td>
<td>---</td>
<td>167.5</td>
<td>34378</td>
<td>500</td>
<td>Intact</td>
</tr>
<tr>
<td>3</td>
<td>1786.0</td>
<td>86</td>
<td>Triax</td>
<td>Quartzite</td>
<td>5</td>
<td>---</td>
<td>173.3</td>
<td>0</td>
<td>-4435</td>
<td>Intact</td>
</tr>
<tr>
<td>4</td>
<td>1617.0</td>
<td>74</td>
<td>Disk</td>
<td>Quartzite</td>
<td>4</td>
<td>---</td>
<td>159.8</td>
<td>0</td>
<td>-2817</td>
<td>Intact</td>
</tr>
<tr>
<td>4</td>
<td>1617.0</td>
<td>74</td>
<td>Triax</td>
<td>Quartzite</td>
<td>4</td>
<td>---</td>
<td>159.0</td>
<td>8409</td>
<td>0</td>
<td>Both</td>
</tr>
<tr>
<td>4</td>
<td>1994.0</td>
<td>95</td>
<td>Triax</td>
<td>Hornfelsy Quartzite</td>
<td>4</td>
<td>---</td>
<td>165.8</td>
<td>0</td>
<td>-2817</td>
<td>Intact</td>
</tr>
<tr>
<td>4</td>
<td>2581.0</td>
<td>95</td>
<td>Disk</td>
<td>Quartzite</td>
<td>4</td>
<td>---</td>
<td>139.7</td>
<td>0</td>
<td>-146</td>
<td>Both</td>
</tr>
<tr>
<td>4</td>
<td>2581.0</td>
<td>95</td>
<td>Triax</td>
<td>Quartzite</td>
<td>4</td>
<td>---</td>
<td>142.9</td>
<td>3704</td>
<td>500</td>
<td>Both</td>
</tr>
<tr>
<td>4</td>
<td>2581.0</td>
<td>95</td>
<td>Uni</td>
<td>Quartzite</td>
<td>2</td>
<td>---</td>
<td>144.1</td>
<td>1407</td>
<td>0</td>
<td>Intact</td>
</tr>
<tr>
<td>5</td>
<td>430.0</td>
<td>97</td>
<td>Disk</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>151.4</td>
<td>0</td>
<td>-673</td>
<td>Both</td>
</tr>
<tr>
<td>5</td>
<td>430.0</td>
<td>97</td>
<td>Uni</td>
<td>Quartzite</td>
<td>2</td>
<td>3</td>
<td>154.1</td>
<td>3337</td>
<td>0</td>
<td>Fracture</td>
</tr>
<tr>
<td>5</td>
<td>622.0</td>
<td>91</td>
<td>SSDS</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>163.9</td>
<td>0</td>
<td>-1570</td>
<td>Both</td>
</tr>
<tr>
<td>5</td>
<td>2224.5</td>
<td>94</td>
<td>Disk</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>164.9</td>
<td>13674</td>
<td>0</td>
<td>Both</td>
</tr>
<tr>
<td>6</td>
<td>97.0</td>
<td>72</td>
<td>Disk</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>158.5</td>
<td>0</td>
<td>-298</td>
<td>Fracture</td>
</tr>
<tr>
<td>6</td>
<td>97.0</td>
<td>72</td>
<td>Uni</td>
<td>Quartzite</td>
<td>2</td>
<td>3</td>
<td>158.4</td>
<td>3221</td>
<td>0</td>
<td>Both</td>
</tr>
<tr>
<td>6</td>
<td>760.0</td>
<td>80</td>
<td>Disk</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>159.9</td>
<td>0</td>
<td>-912</td>
<td>Both</td>
</tr>
<tr>
<td>6</td>
<td>760.0</td>
<td>80</td>
<td>Uni</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>159.1</td>
<td>6141</td>
<td>0</td>
<td>Both</td>
</tr>
<tr>
<td>6</td>
<td>2097.0</td>
<td>72</td>
<td>Disk</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>171.8</td>
<td>0</td>
<td>-1254</td>
<td>Both</td>
</tr>
<tr>
<td>6</td>
<td>2097.0</td>
<td>72</td>
<td>Triax</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>160.4</td>
<td>9946</td>
<td>1000</td>
<td>Intact</td>
</tr>
<tr>
<td>6</td>
<td>4409.0</td>
<td>73</td>
<td>Disk</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>160.9</td>
<td>0</td>
<td>-1749</td>
<td>Both</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>4409.0</td>
<td>73</td>
<td>Triax</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>161.1</td>
<td>21894</td>
<td>1500</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---------</td>
<td>----</td>
<td>-------</td>
<td>-----------</td>
<td>----</td>
<td>----</td>
<td>--------</td>
<td>-------</td>
<td>-------</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>470.0</td>
<td>96</td>
<td>Disk</td>
<td>Quartzite</td>
<td>3</td>
<td>4</td>
<td>163.6</td>
<td>0</td>
<td>-1169</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>470.0</td>
<td>96</td>
<td>Uni</td>
<td>Quartzite</td>
<td>3</td>
<td>4</td>
<td>162.7</td>
<td>7092</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>514.0</td>
<td>98</td>
<td>Disk</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>161.3</td>
<td>0</td>
<td>-812</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>514.0</td>
<td>98</td>
<td>Uni</td>
<td>Quartzite</td>
<td>4</td>
<td>3</td>
<td>161.7</td>
<td>8408</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>746.0</td>
<td>94</td>
<td>Disk</td>
<td>Quartzite</td>
<td>3</td>
<td>4</td>
<td>165.3</td>
<td>0</td>
<td>-1345</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>746.0</td>
<td>94</td>
<td>Uni</td>
<td>Quartzite</td>
<td>3</td>
<td>4</td>
<td>160.2</td>
<td>6630</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>645.0</td>
<td>90</td>
<td>Disk</td>
<td>Quartzite</td>
<td>4</td>
<td>4</td>
<td>164.3</td>
<td>0</td>
<td>-1361</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>645.0</td>
<td>90</td>
<td>Uni</td>
<td>Quartzite</td>
<td>4</td>
<td>4</td>
<td>163.7</td>
<td>9913</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>645.0</td>
<td>83</td>
<td>Disk</td>
<td>Quartzite</td>
<td>4</td>
<td>4</td>
<td>164.2</td>
<td>0</td>
<td>-1879</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>645.0</td>
<td>83</td>
<td>Uni</td>
<td>Quartzite</td>
<td>4</td>
<td>4</td>
<td>165.2</td>
<td>9778</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>9</td>
<td>364.0</td>
<td>57</td>
<td>Disk</td>
<td>Hornfelsy Quartzite</td>
<td>4</td>
<td>3</td>
<td>160.7</td>
<td>0</td>
<td>-4628</td>
</tr>
<tr>
<td>9</td>
<td>9</td>
<td>364.0</td>
<td>57</td>
<td>Uni</td>
<td>Hornfelsy Quartzite</td>
<td>6</td>
<td>3</td>
<td>165.6</td>
<td>37852</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>9</td>
<td>541.0</td>
<td>39</td>
<td>Disk</td>
<td>Quartzite</td>
<td>4</td>
<td>3</td>
<td>162.3</td>
<td>0</td>
<td>-851</td>
</tr>
<tr>
<td>9</td>
<td>9</td>
<td>541.0</td>
<td>39</td>
<td>Triax</td>
<td>Quartzite</td>
<td>4</td>
<td>3</td>
<td>164.3</td>
<td>15440</td>
<td>1000</td>
</tr>
<tr>
<td>9</td>
<td>9</td>
<td>2016.0</td>
<td>38</td>
<td>Disk</td>
<td>Quartzite</td>
<td>4</td>
<td>3</td>
<td>194.4</td>
<td>0</td>
<td>-1189</td>
</tr>
<tr>
<td>9</td>
<td>9</td>
<td>2016.0</td>
<td>38</td>
<td>Uni</td>
<td>Quartzite</td>
<td>3</td>
<td>3</td>
<td>187.2</td>
<td>7120</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>442.0</td>
<td>99</td>
<td>Disk</td>
<td>Hornfelsy</td>
<td>3</td>
<td>3</td>
<td>167.6</td>
<td>0</td>
<td>-760</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>442.0</td>
<td>99</td>
<td>Triax</td>
<td>Hornfelsy</td>
<td>3</td>
<td>3</td>
<td>168.2</td>
<td>25381</td>
<td>2000</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>480.0</td>
<td>83</td>
<td>Disk</td>
<td>Hornfelsy</td>
<td>3</td>
<td>2</td>
<td>164.3</td>
<td>0</td>
<td>-515</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>480.0</td>
<td>83</td>
<td>Uni</td>
<td>Hornfelsy</td>
<td>2</td>
<td>2</td>
<td>169.6</td>
<td>2415</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>760.0</td>
<td>90.8</td>
<td>SSDS</td>
<td>Hornfelsy</td>
<td>3</td>
<td>2</td>
<td>----</td>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>800.0</td>
<td>100</td>
<td>Disk</td>
<td>Hornfelsy</td>
<td>3</td>
<td>3</td>
<td>160.8</td>
<td>0</td>
<td>-1159</td>
</tr>
<tr>
<td>11</td>
<td>11</td>
<td>800.0</td>
<td>100</td>
<td>Triax</td>
<td>Hornfelsy</td>
<td>3</td>
<td>3</td>
<td>164.3</td>
<td>15504</td>
<td>500</td>
</tr>
<tr>
<td>11</td>
<td>11</td>
<td>134.0</td>
<td>77</td>
<td>SSDS</td>
<td>Hornfelsy</td>
<td>4</td>
<td>4</td>
<td>----</td>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td>11</td>
<td>11</td>
<td>272.0</td>
<td>87</td>
<td>Disk</td>
<td>Quartzite</td>
<td>4</td>
<td>4</td>
<td>159.9</td>
<td>0</td>
<td>-1660</td>
</tr>
<tr>
<td>11</td>
<td>11</td>
<td>272.0</td>
<td>87</td>
<td>Triax</td>
<td>Quartzite</td>
<td>4</td>
<td>4</td>
<td>162.3</td>
<td>21994</td>
<td>1000</td>
</tr>
<tr>
<td>11</td>
<td>11</td>
<td>1801.0</td>
<td>100</td>
<td>Disk</td>
<td>Quartzite</td>
<td>4</td>
<td>4</td>
<td>181.8</td>
<td>0</td>
<td>-1930</td>
</tr>
<tr>
<td>11</td>
<td>11</td>
<td>1801.0</td>
<td>100</td>
<td>Uni</td>
<td>Quartzite</td>
<td>4</td>
<td>4</td>
<td>173.9</td>
<td>13593</td>
<td>0</td>
</tr>
<tr>
<td>12</td>
<td>12</td>
<td>206.0</td>
<td>18</td>
<td>Disk</td>
<td>Quartzite</td>
<td>0</td>
<td>4</td>
<td>160.1</td>
<td>0</td>
<td>-1806</td>
</tr>
<tr>
<td>12</td>
<td>12</td>
<td>903.0</td>
<td>18</td>
<td>Uni</td>
<td>Quartzite</td>
<td>0</td>
<td>4</td>
<td>152.0</td>
<td>0</td>
<td>-25</td>
</tr>
<tr>
<td>12</td>
<td>12</td>
<td>903.0</td>
<td>18</td>
<td>Disk</td>
<td>Quartzite</td>
<td>0</td>
<td>4</td>
<td>153.4</td>
<td>60</td>
<td>0</td>
</tr>
</tbody>
</table>
REFERENCES


Call & Nicholas, Inc. 2006. Relationship Between Hardness, Consistency and Uniaxial Compressive Strength Table. Tucson, AZ USA


Cooley, L.A, Jr., Huner, M.S., James, R.S. (2002). Micro-Deval testing of aggregates in the southeast. National Center for Asphalt Technology, Auburn University. NCAT Report No. 02-09


Pei, T., Qin, C., Zhu, A., Yang, L., Li, B., 2007, Mapping Soil Carbon Using Collocated Cokriging with Wetness Index, 12th Conference of Int. Association for Mathematical Geology


Qin, C., Zhu, A., Yang, L., Li, B., Pei, T., 2006, Topographic Wetness Index Computed Using Multiple Flow Direction Algorithm and Local Maximum Downslope Gradient, the 7th International Workshop of Geographical Information System


Trauth, Martin H. 2006, Matlab Recipes for Earth Sciences, New York: Springer.

Veerle Vanacker, Michiel Vanderschaeghe, Gerard Govers, Edith Willems, Jean Poesen, Jozef Deckers, Bert De Bievre, Linking hydrological, infinite slope stability
and land-use change models through GIS for assessing the impact of deforestation on slope stability in high Andean watersheds, Geomorphology, Volume 52, Issues 3-4, 16 June 2003, Pages 299-315

Walnut Gulch Watershed Brochure, 2007, Southwest Watershed Research Center


WRES, Inc. Pumping consulting services. Slurry abrasivity determination. Frisco, TX USA. http://www.wres.us/Home.aspx

