EFFECT OF DELAYED BACKFILL ON OPEN STOPE MINING METHODS

by

CRISTIAN ANDRES CACERES DOERNER

B.Sc., Universidad de Chile, 1997

A THESIS SUBMITTED IN PARTIAL FULFILMENT OF

THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

in

THE FACULTY OF GRADUATE STUDIES

MINING ENGINEERING

THE UNIVERSITY OF BRITISH COLUMBIA

March 2005

© Cristian Andres Caceres Doerner, 2005
Open stoping is comprised of large rectangular voids separated by intervening pillars so as to minimize the size of the exposed surface and thereby reducing the potential for wall slough and in turn external dilution.

These pillars provide support to the exposed wall; however, they result in ore loss and increased costs such as having to establish slots for blasting. Longhole mining methods such as Avoca or longhole retreat as practiced at the Musselwhite mine of Placer Dome employs 100% extraction with the use of fill walls to provide support to the adjacent stope.

Transverse open stoping also practiced at Musselwhite employs cemented rock fill adjacent to a mined stope. The question is how to account for the overall stope wall stability when the adjacent support is backfill. The backfill does not provide the same support as that of a rock pillar, however, due to the increased use of fill abutments one has to develop a methodology that accounts for this reduced overall support element as it does reduce the overall stope surface exposed.

It has been shown in this thesis that the backfill wall does not provide the same overall stability to an individual stope as would avail itself if the stope had rock abutments.

This is the focus of this study in order to establish design criteria to enable one to employ existing methods for stope design such as the Stability Graph by augmenting input parameters that have been calibrated through field measurements, analytical assessments, numerical modeling and laboratory testing to evaluate the effect of mining adjacent to a backfilled stope.

Sill pillars are employed at the Musselwhite mine to allow for multiple mining horizons with unconsolidated backfill placed immediately above the intervening sill. These sills can be comprised of unmined ore when the economics are such as to negate their mining or alternatively they are replaced by a constructed sill mat to allow for mining underneath by non-entry methods and thereby containing the overlying backfill.

Numerical studies were conducted to investigate modeled results of mining under a cemented rockfill sill mat and to develop criteria for sill mat design. The results obtained from this analysis can be extrapolated to other operations that utilize backfill as part of the
mining sequence. Design curves were developed for the stability of sill mats for various stope configurations and cemented rockfill strength properties.

This, coupled with defining the effect of mining adjacent to backfill, forms the focus of this thesis.
# TABLE OF CONTENTS

**ABSTRACT** ........................................................................................................... II

**TABLE OF CONTENTS** ....................................................................................... IV

**LIST OF TABLES** ............................................................................................. VII

**TABLE OF FIGURES** ........................................................................................ VIII

**ACKNOWLEDGMENTS** ....................................................................................... XI

1 **INTRODUCTION** ............................................................................................ 1

1.3 **BACKGROUND** ......................................................................................... 2

1.4 **THESIS OVERVIEW** .................................................................................. 3

1.5 **CONTRIBUTIONS MADE BY THESIS** .................................................... 3

2 **THE MUSSELWHITE MINE** .......................................................................... 5

2.1 **INTRODUCTION** ......................................................................................... 5

2.2 **GEOLOGY** .................................................................................................. 6

2.2.1 **Regional Geology** .................................................................................. 6

2.2.2 **Mine Geology** ......................................................................................... 7

2.3 **UNDERGROUND MINING METHODS** ....................................................... 9

2.3.1 **Avoca** ..................................................................................................... 9

2.3.2 **Transverse Retreat Open Stopping** ......................................................... 10

2.3.3 **Drilling and Blasting Methods** ............................................................... 10

2.3.4 **Ore Handling** ........................................................................................ 11

2.3.5 **Ground Support** .................................................................................... 11

2.3.5.1 Development Support ............................................................................ 12

2.3.5.2 Stope support – Stope backs and walls ............................................... 12

2.3.6 **Backfill** ................................................................................................ 13

2.3.6.1 Mining Underneath Backfill – CRF Sill Mats ..................................... 14

2.3.6.2 Mining Adjacent to Backfill ................................................................. 14

2.4 **EQUIPMENT** .............................................................................................. 16

3 **REVIEW OF DESIGN METHODOLOGIES** ............................................ 17

3.1 **BACKFILL** ............................................................................................... 17

3.1.1 **Purpose of Backfill** ............................................................................... 17

3.1.2 **Types of Backfill** .................................................................................. 19

3.1.2.1 Rockfill .................................................................................................. 20

3.1.2.2 Uncemented Rockfill – URF ................................................................. 20

3.1.2.3 Cemented Rockfill with Portland cement Slurry – CRF ..................... 21

3.1.3 **Backfill Properties** ............................................................................... 21

3.1.4 **Parameters Affecting Cemented Rockfill Strength** ............................. 26

3.2 **VERTICAL LOAD ON CEMENTED ROCKFILL SILL MAT PILARS** .... 28

3.2.1 **Sill Mat Pillar Definition** ....................................................................... 28

3.2.2 **Background – Loads Acting by Unconsolidated Material** .................. 29

3.2.3 **Maximum Horizontal Pressure on the Walls of the Stope** ................. 31

3.2.4 **Coefficient of Lateral Earth Pressure** .................................................. 33

3.2.4.1 Categories of Lateral Earth Pressure .................................................... 33

3.2.4.2 Calculating Lateral Earth Pressure Coefficients ................................. 35

3.2.5 **Maximum Vertical Load Exerted by Backfill on the Floor of the Stope** 38

3.2.5.1 Terzaghi's Formulation ........................................................................ 39

3.2.5.2 Blight's Formulation for Inclined Stopes ............................................ 39

3.2.6 **Vertical Load of Unconsolidated Material as a Function of the Material's Height** 40

3.2.6.1 The Janssen Method – Silo Theory ..................................................... 40

3.2.6.2 The Reimbert Method – Silo Theory ................................................... 41

3.3 **STABILITY OF CEMENTED ROCKFILL SILL MATS – ANALYTICAL SOLUTION** 42
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.3.1</td>
<td>Caving Failure</td>
<td>42</td>
</tr>
<tr>
<td>3.3.2</td>
<td>Flexural Failure</td>
<td>43</td>
</tr>
<tr>
<td>3.3.3</td>
<td>Sliding Failure</td>
<td>44</td>
</tr>
<tr>
<td>3.3.4</td>
<td>Rotational Failure</td>
<td>44</td>
</tr>
<tr>
<td>3.3.5</td>
<td>Strength Properties</td>
<td>44</td>
</tr>
<tr>
<td>3.4</td>
<td><strong>Empirical Stability Design Methods</strong></td>
<td>45</td>
</tr>
<tr>
<td>3.4.1</td>
<td>The Stability Graph Method</td>
<td>46</td>
</tr>
<tr>
<td>3.4.2</td>
<td>The Stability Graph Method - Radius Factor</td>
<td>48</td>
</tr>
<tr>
<td>3.5</td>
<td>Software and Device Utilized</td>
<td>49</td>
</tr>
<tr>
<td>3.5.1</td>
<td>FLAC^2D - Two Dimensional Explicit Finite Difference Method</td>
<td>49</td>
</tr>
<tr>
<td>3.5.2</td>
<td>Map3D - Three Dimensional Boundary Element Method</td>
<td>50</td>
</tr>
<tr>
<td>3.5.3</td>
<td>NeuroShell Predictor - Neural Networks</td>
<td>51</td>
</tr>
<tr>
<td>3.5.4</td>
<td>Unwedge - Underground Wedge Stability Analysis</td>
<td>52</td>
</tr>
<tr>
<td>3.5.5</td>
<td>Dips - Graphical and Statistical Analysis of Orientation Data</td>
<td>53</td>
</tr>
<tr>
<td>3.5.6</td>
<td>Methods Ground Control Assessment - Cavity Monitoring System Surveys</td>
<td>53</td>
</tr>
<tr>
<td>4.1</td>
<td>Database</td>
<td>55</td>
</tr>
<tr>
<td>4.2</td>
<td><strong>Intact Strength</strong></td>
<td>55</td>
</tr>
<tr>
<td>4.3</td>
<td><strong>Fabric Analysis</strong></td>
<td>56</td>
</tr>
<tr>
<td>4.4</td>
<td><strong>Rock Mass Analysis</strong></td>
<td>58</td>
</tr>
<tr>
<td>4.5</td>
<td>Numerical Modeling - Constitutive Models and Material Properties</td>
<td>60</td>
</tr>
<tr>
<td>4.6</td>
<td><strong>Empirical Stope Design</strong> - Musselwhite's Stability Database</td>
<td>61</td>
</tr>
<tr>
<td>4.7</td>
<td>Statistical Analysis of the Musselwhite Database</td>
<td>63</td>
</tr>
<tr>
<td>5.1</td>
<td>Design Guidelines for Cemented Rockfill Sill Mats</td>
<td>66</td>
</tr>
<tr>
<td>5.2</td>
<td><strong>Introduction</strong></td>
<td>66</td>
</tr>
<tr>
<td>5.2.1</td>
<td>Vertical Stress Comparison using Different Coefficients of Lateral Earth Pressure</td>
<td>70</td>
</tr>
<tr>
<td>5.3</td>
<td><strong>Vertical Stress Using Numerical Modeling</strong></td>
<td>71</td>
</tr>
<tr>
<td>5.3.1</td>
<td>Introduction</td>
<td>71</td>
</tr>
<tr>
<td>5.3.2</td>
<td>Stress Distribution as a Function of Stope Dip Angle</td>
<td>74</td>
</tr>
<tr>
<td>5.3.3</td>
<td>Vertical Stress as a Function of Stope Span</td>
<td>75</td>
</tr>
<tr>
<td>5.3.4</td>
<td>Vertical Stress as a Function of Rockfill's Density</td>
<td>76</td>
</tr>
<tr>
<td>5.3.5</td>
<td>Vertical Stress as a Function of Rockfill's Friction Angle</td>
<td>76</td>
</tr>
<tr>
<td>5.3.6</td>
<td>Comparison of Analytical and Numerical Methods to Determine Vertical Stress</td>
<td>77</td>
</tr>
<tr>
<td>5.3.7</td>
<td>Proposed Analytical Equation for Inclined Stopes</td>
<td>79</td>
</tr>
<tr>
<td>5.4</td>
<td>Stability of Cemented Rockfill Sill Mats</td>
<td>80</td>
</tr>
<tr>
<td>5.4.1</td>
<td>Introduction</td>
<td>80</td>
</tr>
<tr>
<td>5.4.2</td>
<td>Sill Mat Strength - Friction and Cohesion (φ)</td>
<td>81</td>
</tr>
<tr>
<td>5.4.3</td>
<td>Backfill Load</td>
<td>82</td>
</tr>
<tr>
<td>5.4.4</td>
<td>Proposed Rotational Analytical Formulation</td>
<td>82</td>
</tr>
<tr>
<td>5.5</td>
<td>Failure Modes - Numerical Modeling Solution</td>
<td>85</td>
</tr>
<tr>
<td>5.5.1</td>
<td>Model Construction</td>
<td>85</td>
</tr>
<tr>
<td>5.5.2</td>
<td>Constitutive Equations - Strain Softening</td>
<td>86</td>
</tr>
<tr>
<td>5.5.3</td>
<td>Model Execution</td>
<td>87</td>
</tr>
<tr>
<td>5.5.4</td>
<td>Caving Failure</td>
<td>88</td>
</tr>
<tr>
<td>5.5.5</td>
<td>Flexural Failure</td>
<td>88</td>
</tr>
<tr>
<td>5.5.6</td>
<td>Sliding Failure</td>
<td>89</td>
</tr>
<tr>
<td>5.5.7</td>
<td>Rotational - Crushing Failure</td>
<td>90</td>
</tr>
<tr>
<td>5.5.8</td>
<td>Rotational - Breaking Failure</td>
<td>90</td>
</tr>
<tr>
<td>5.5.9</td>
<td>Sill Mat Design Curves</td>
<td>91</td>
</tr>
<tr>
<td>6.1</td>
<td>Effect of Delayed Backfill on Open Stopping</td>
<td>97</td>
</tr>
<tr>
<td>6.2</td>
<td>Introduction</td>
<td>97</td>
</tr>
<tr>
<td>6.2</td>
<td>Backfill As Local Support</td>
<td>97</td>
</tr>
</tbody>
</table>
## Table of Contents

6.3 **USE OF THE STABILITY GRAPH METHOD** ................................................................. 99
6.4 **QUANTIFYING THE EFFECT OF BACKFILL ON LONGHOLE OPEN STOPING** ............ 100
6.5 **UPDATING MUSSELWHITE'S STABILITY GRAPH** ................................................. 100
6.6 **NEURAL NETWORK TRAINING AND RESULTS** ..................................................... 104
6.6.1 Relative Importance of Inputs ............................................................................. 104
6.6.2 Neural Network Predictions .............................................................................. 107
6.7 **EFFECT OF BACKFILL IN STABILITY OF OPEN STOPES – NUMERICAL MODELING ANALYSIS** .......................................................... 110
6.7.1 Radius Factor Behavior with Stope Strike Length ............................................. 112
6.7.2 Horizontal Displacement Behavior considered with Stope Strike Length .......... 115
6.7.3 Design Curves – Avoca Mining Method .............................................................. 116

7 **CONCLUSIONS AND RECOMMENDATIONS** ...................................................... 120

8 **FUTURE WORK** ................................................................................................... 125

REFERENCES ............................................................................................................. 128

APPENDIX A ............................................................................................................... 134

PROPOSED ANALYTICAL EQUATION TO DETERMINE VERTICAL LOAD OF BACKFILL ... 134

APPENDIX B ............................................................................................................... 138

PROPOSED ANALYTICAL EQUATION TO DETERMINE ROTATIONAL SILL MAT FAILURE MODES ........................................................................................................... 138

APPENDIX C ............................................................................................................... 139

INTERFACE ELEMENTS.............................................................................................. 139
LIST OF TABLES

Table 2-1: Support strength properties .......................................................... 11
Table 2-2: Development support ....................................................................... 12
Table 2-3: Mobile Equipment List ...................................................................... 16
Table 4-1: Mean uniaxial compressive strength, standard deviation and ISRM hardness .............................................................................................................. 55
Table 4-2: Mean Young's Modulus (E) and Mean Poisson's Ratio (v) .................. 56
Table 4-3: RMR and Q' of a typical footwall stope ............................................. 59
Table 4-4: Typical RMR ranges for the different Musselwhite rock types .......... 59
Table 4-5: Musselwhite's far field stress state .................................................... 60
Table 4-6: Mohr-Coulomb stress-strain and strength parameters for backfill and host rock ............................................................................................................. 61
Table 4-7: Updated Musselwhite stability database .............................................. 63
Table 5-1: Example of Musselwhite's geometry, rockfill loading, and sill mat strength properties ........................................................................................................ 86
Table 6-1: Average and standard deviation for the Musselwhite's stability database. 102
# Table of Figures

<p>| Figure 2-1: Location of Musselwhite mine | ......................................................... | 5    |
| Figure 2-2: Long section of Musselwhite orebody | ......................................................... | 5    |
| Figure 2-3: Geological cross-section through the T-Antiform Northern Iron Formation | ......................................................... | 6    |
| Figure 2-4: Geological cross-section showing the Wa, T, C and S zones | ......................................................... | 7    |
| Figure 2-5: Longitudinal retreat open stoping -- Mining adjacent to backfill | ......................................................... | 9    |
| Figure 2-6: Transverse open stoping -- Mining adjacent to backfill | ......................................................... | 10   |
| Figure 2-7: Cemented rockfill sill mats constructed: 275, 300 and 375 m Levels | ......................................................... | 14   |
| Figure 3-1: Purposes of backfilling | ......................................................... | 18   |
| Figure 3-2: Mohr-Coulomb failure envelope for a cohesive fill material | ......................................................... | 24   |
| Figure 3-3: Mohr-Coulomb failure envelope for a cohesionless fill material | ......................................................... | 25   |
| Figure 3-4: Angle of repose $\phi$ for cohesionless fill material | ......................................................... | 26   |
| Figure 3-5: Sill mat pillar cross-section | ......................................................... | 28   |
| Figure 3-6: Natural angle of repose of the material (left) and the oblique, normal and tangential components of the force (right) | ......................................................... | 29   |
| Figure 3-7: Vertical stress at increasing depth for a confined material | ......................................................... | 31   |
| Figure 3-8: Pressure on a horizontal slice of thickness $\delta z$ | ......................................................... | 31   |
| Figure 3-9: Relationship of earth pressures to wall movements | ......................................................... | 35   |
| Figure 3-10: Comparison of at-rest earth and active earth pressure coefficients | ......................................................... | 35   |
| Figure 3-11: Differential slice in a silo | ......................................................... | 40   |
| Figure 3-12: Mathews Stability Graph (Mathews et al., 1981) | ......................................................... | 46   |
| Figure 3-13: Modified Stability Graph (Potvin and Milne, 1992) | ......................................................... | 47   |
| Figure 3-14: Modified Stability Graph with support (Nickson, 1992) | ......................................................... | 47   |
| Figure 3-15: ELOS dilution design method (Clark, 1998) | ......................................................... | 48   |
| Figure 3-16: Empirical estimation (a) of wall slough (ELOS) expressed in terms of Radius Factor (b) | ......................................................... | 49   |
| Figure 4-1: Equal area stereonet representing the three major joint sets | ......................................................... | 57   |
| Figure 4-2: Wedge formed on the back of a drift running north-south | ......................................................... | 58   |
| Figure 4-3: Geological section obtained using Vulcan database | ......................................................... | 60   |
| Figure 4-4: Statistical Analysis on Musselwhite stability database | ......................................................... | 64   |
| Figure 5-1: Coefficient of lateral earth pressure $K$ for cohesionless material | ......................................................... | 67   |
| Figure 5-2: Comparison of analytically and numerically determined coefficients of lateral earth pressure | ......................................................... | 68   |
| Figure 5-3: Coefficient of lateral earth pressure obtained using numerical modeling and best fit curve derived | ......................................................... | 69   |
| Figure 5-4: Coefficient of lateral earth pressure obtained using numerical modeling and best fit curve derived for the rockfill friction angle range | ......................................................... | 69   |
| Figure 5-5: Horizontal vs. vertical stress at different rockfill friction angles | ......................................................... | 70   |
| Figure 5-6: Vertical stress comparison for different $K$ values | ......................................................... | 71   |
| Figure 5-7: 25 meters (5 layers) and 35 meters (7 layers) of unconsolidated backfill deposited in the stope | ......................................................... | 73   |
| Figure 5-8: Vertical stress distribution along span at different stope dips | ......................................................... | 74   |
| Figure 5-9: Stress contour for a stope dipping at 70 degrees | ......................................................... | 75   |
| Figure 5-10: Vertical stress at various stope spans for different rockfill heights | ......................................................... | 75   |
| Figure 5-11: Vertical stress vs. rockfill density at varying rockfill heights | ......................................................... | 76   |</p>
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-12</td>
<td>Vertical stress vs. rockfill height at varying friction angles</td>
<td>77</td>
</tr>
<tr>
<td>5-13</td>
<td>Vertical stress over rockfill height comparing analytical and numerical results</td>
<td>78</td>
</tr>
<tr>
<td>5-14</td>
<td>Vertical stress versus stope span using different methods</td>
<td>78</td>
</tr>
<tr>
<td>5-15</td>
<td>Vertical stress for a 10 meter stope span using different methods</td>
<td>79</td>
</tr>
<tr>
<td>5-16</td>
<td>Analytical vs. numerical modeling comparison for different stope dip angles</td>
<td>80</td>
</tr>
<tr>
<td>5-17</td>
<td>Geometry, strength and stress component and failure modes in a sill mat pillar</td>
<td>81</td>
</tr>
<tr>
<td>5-18</td>
<td>Rotational failure considering shear strength in the hangingwall of the sill mat</td>
<td>83</td>
</tr>
<tr>
<td>5-19</td>
<td>Rotational failure considering shear strength in the hangingwall of the sill mat</td>
<td>84</td>
</tr>
<tr>
<td>5-20</td>
<td>Factor of safety for sill mat failure modes</td>
<td>84</td>
</tr>
<tr>
<td>5-21</td>
<td>Strain softening model – cohesion example</td>
<td>87</td>
</tr>
<tr>
<td>5-22</td>
<td>Caving failure mode – planar and semi circular crack</td>
<td>88</td>
</tr>
<tr>
<td>5-23</td>
<td>Sill mat flexural failure mode – grid elements and plasticity state</td>
<td>89</td>
</tr>
<tr>
<td>5-24</td>
<td>Sill mat sliding failure mode – grid elements and plasticity state</td>
<td>89</td>
</tr>
<tr>
<td>5-25</td>
<td>Sill mat rotational crushing failure mode – grid elements and plasticity state</td>
<td>90</td>
</tr>
<tr>
<td>5-26</td>
<td>Sill mat rotational breaking failure mode – grid elements and plasticity state</td>
<td>91</td>
</tr>
<tr>
<td>5-27</td>
<td>Sill mat stability for 90° – No strength on HW (τ_t = 0%)</td>
<td>92</td>
</tr>
<tr>
<td>5-28</td>
<td>Sill mat stability for 90° – 50% sill mat strength on HW (τ_t = 50%)</td>
<td>92</td>
</tr>
<tr>
<td>5-29</td>
<td>Sill mat stability for 90° – 100% sill mat strength on HW (τ_t = 100%)</td>
<td>92</td>
</tr>
<tr>
<td>5-30</td>
<td>Sill mat stability for 85° – No strength on HW (τ_t = 0%)</td>
<td>93</td>
</tr>
<tr>
<td>5-31</td>
<td>Sill mat stability for 85° – 50% sill mat strength on HW (τ_t = 50%)</td>
<td>93</td>
</tr>
<tr>
<td>5-32</td>
<td>Sill mat stability for 85° – 100% sill mat strength on HW (τ_t = 100%)</td>
<td>93</td>
</tr>
<tr>
<td>5-33</td>
<td>Sill mat stability for 80° – No strength on HW (τ_t = 0%)</td>
<td>94</td>
</tr>
<tr>
<td>5-34</td>
<td>Sill mat stability for 80° – 50% sill mat strength on HW (τ_t = 50%)</td>
<td>94</td>
</tr>
<tr>
<td>5-35</td>
<td>Sill mat stability for 80° – 100% sill mat strength on HW (τ_t = 100%)</td>
<td>94</td>
</tr>
<tr>
<td>5-36</td>
<td>Sill mat stability for 75° – No strength on HW (τ_t = 0%)</td>
<td>95</td>
</tr>
<tr>
<td>5-37</td>
<td>Sill mat stability for 75° – 50% sill mat strength on HW (τ_t = 50%)</td>
<td>95</td>
</tr>
<tr>
<td>5-38</td>
<td>Sill mat stability for 75° – 100% sill mat strength on HW (τ_t = 100%)</td>
<td>95</td>
</tr>
<tr>
<td>6-1</td>
<td>Horizontal stress versus rockfill height at different stope spans</td>
<td>98</td>
</tr>
<tr>
<td>6-6</td>
<td>Relative importance of inputs on stope stability for stopes dipping more than 90 degrees (footwalls)</td>
<td>103</td>
</tr>
<tr>
<td>6-7</td>
<td>Relative importance of inputs on stope stability for stope dips under 90 degrees (hangingwalls)</td>
<td>104</td>
</tr>
<tr>
<td>6-8</td>
<td>Relative importance of inputs on stope stability considering rock type, ore dip and undercut</td>
<td>105</td>
</tr>
<tr>
<td>6-9</td>
<td>Stability Graph update – longitudinal stope data (AVOCA)</td>
<td>101</td>
</tr>
<tr>
<td>6-10</td>
<td>Stability Graph update – transverse stope data</td>
<td>102</td>
</tr>
<tr>
<td>6-11</td>
<td>Stability Graph update – transverse and longitudinal stope data</td>
<td>103</td>
</tr>
<tr>
<td>6-12</td>
<td>Relative importance of inputs on stope stability for stopes dipping more than 90 degrees (footwalls)</td>
<td>104</td>
</tr>
<tr>
<td>6-13</td>
<td>Relative importance of inputs on stope stability for stope dips under 90 degrees (hangingwalls)</td>
<td>105</td>
</tr>
<tr>
<td>6-14</td>
<td>Relative importance of inputs on stope stability considering rock type, ore dip and undercut</td>
<td>106</td>
</tr>
<tr>
<td>Figure 6-9: Effect of undercutting the hangingwall: various types of failure</td>
<td>107</td>
<td></td>
</tr>
<tr>
<td>Figure 6-10: Predicted ELOS values of transverse vs. longitudinal stopes</td>
<td>108</td>
<td></td>
</tr>
<tr>
<td>Figure 6-11: Wall instability due to structure on hangingwall</td>
<td>109</td>
<td></td>
</tr>
<tr>
<td>Figure 6-12: Sequence without backfill</td>
<td>110</td>
<td></td>
</tr>
<tr>
<td>Figure 6-13: Sequence with backfill</td>
<td>110</td>
<td></td>
</tr>
<tr>
<td>Figure 6-14: Mining of the top stope only</td>
<td>111</td>
<td></td>
</tr>
<tr>
<td>Figure 6-15: Horizontal displacement profile</td>
<td>111</td>
<td></td>
</tr>
<tr>
<td>Figure 6-16: Schematic of a stope of increasingly larger strike length</td>
<td>112</td>
<td></td>
</tr>
<tr>
<td>Figure 6-17: Radius factor measurement at distance from front face</td>
<td>113</td>
<td></td>
</tr>
<tr>
<td>Figure 6-18: Radius factor generation</td>
<td>114</td>
<td></td>
</tr>
<tr>
<td>Figure 6-19: Radius factor versus stope strike length at different distance from front face</td>
<td>114</td>
<td></td>
</tr>
<tr>
<td>Figure 6-20: Horizontal displacement measurement at various distances from front face</td>
<td>115</td>
<td></td>
</tr>
<tr>
<td>Figure 6-21: Horizontal displacement versus stope strike length at different distances from front face</td>
<td>116</td>
<td></td>
</tr>
<tr>
<td>Figure 6-22: Radius factor increment – stope of 30 meter height and 400 meter strike length</td>
<td>117</td>
<td></td>
</tr>
<tr>
<td>Figure 6-23: Radius factor increments for a stope of 30 meter height and 400 meter strike length</td>
<td>118</td>
<td></td>
</tr>
<tr>
<td>Figure 6-24: Radius factor increments for a stope of 40 meter height and 400 meter strike length</td>
<td>119</td>
<td></td>
</tr>
</tbody>
</table>
ACKNOWLEDGMENTS

I would like to extend my deepest gratitude to all those who have helped to make this thesis possible.

For his assistance and on-going support, and for giving me the opportunity to create this thesis, I thank first and foremost Dr. Rimas Pakalnis. His input and influence represent the sine qua non of my work and will never be forgotten.

Enrique Rubio enthusiastically encouraged me to pursue this degree and I could never fully express my debt of gratitude to him. Thank you beyond words.

Many kind individuals graciously leant their support and the benefit of their experience, including Robert McDonald, Rod Gray, Tim Sanford, Don Peterson, Joe Hunter, Ken Strobbe, Jean-Marc Dallaire, Cameron Chapman, Sam Mah, and other personnel of Placer Dome’s Musselwhite mining operation that I may be remiss in not naming specifically. Forgive me.

For their instrumental recommendations and counsel, I extend my sincere thanks to committee members, Dr. Malcolm Scoble, Dr. Mario Morin, and David Sprott.

I am enormously grateful for the financial support provided to me by the Department of Mining and Mineral Processing Engineering at UBC, as well as by NSERC and Placer Dome’s Musselwhite Mine. Without this support, no thesis, indeed no career, would be possible.

I would also like to extend my sincerest appreciation to my parents for their continuous support and encouragement during my studies.

I am infinitely thankful to my patient, forgiving, and un-selfishly supportive wife Veronica, and to my inspiration to do all that I am capable of doing, my precious daughter Victoria.

For pointing out errors and omissions, and tactfully criticizing the presentation of my ideas, research, and theoretical conclusions on paper, I would also like to thank my editor, Trenton McColl.
Chapter 1 – Introduction

1 INTRODUCTION

1.1 Preface

Research into design methodologies for open stope mining is essential to overall mine efficiency and productivity. New, more accurate, design guidelines will prove more cost effective than those in existence, thus effecting a more competitive mining operation. Additionally, these new guidelines will reduce mine waste while improving safety conditions.

To arrive at new design guidelines, the first component in stope design to be considered is analysis of cemented rockfill, for the loading case (i.e., loading of uncemented rockfill material), and for failure mode study (i.e., the limits of the design).

The second component is analysis of backfill adjacent to open workings (i.e., stopes) and its influence on design.

A method based upon past practice and observed measurements (combined with modeling analysis) will be employed to arrive at a new methodology that accounts for an increase in overall stability.

1.2 Open Stopes

Open stoping accounts for over 60% (De Souza, et al., 1998) of the total underground tonnage in Canadian mining operations.

Open stoping in Canada is one of the most cost effective and productive underground mining methods. This thesis focuses on the geomechanics of mining by longhole open stoping, adjacent to backfill. The objective is to improve recovery and reduce dilution. Dilution and recovery are crucial factors, and may determine the viability of a given mining operation. A mine may fall below profitability where dilution is higher than expected or recovery is lower than predicted, because of excess ore left behind in the form of sills and/or rib pillars. These factors are investigated in detail.

All field research was completed at Placer Dome’s Musselwhite mine. Musselwhite is a 4000 tpd, underground gold mine, located 500 km north of Thunder Bay, Ontario.

The underground methods in use at Musselwhite are longitudinal retreat and/or transverse open stope mining, and voids are subsequently backfilled using development
waste rock and open pit waste rock. The waste is augmented by adding cement, and used for either backfilling primary stopes or in sill mat construction. Chapter 4 details these activities.

In the first case (Chapter 5), the research focuses on sill mat failure modes by analyzing current conventional methods (re: Musselwhite). The second case (Chapter 6) concentrates on rock behavior when mining adjacent to backfilled stopes (primary, and indirectly, secondary stopes) and where longitudinal retreat with delayed backfill is employed. The effect of backfill on longitudinal retreat mining (Avoca) is examined in light of its influence on stability design.

Empirical tools, numerical modeling, and equilibrium analyses are supplemented by field data, illustrating how backfill affects the stability of mining adjacent to, and below, a filled stope.

1.3 Background

Mining open stopes in the proximity of backfill has become an increasingly popular mining method over the past 10 years. This is principally due to recent technical innovations such as the introduction of pastefill and cemented rockfill. In the past, cemented hydraulic stopes had less capacity for quality control compared with that of recent innovations in stope design. Various methods of sizing stopes have evolved since 1982, when Mathews introduced the “Stability Graph”. However, these methods focus on isolated stopes, which have solid pillar abutments along the strike/dip.

Substitution of backfill for a rock pillar abutment intuitively implies compromised overall stability of the open stope. This thesis addresses this concern and presents an updated amendment to the “Stability Graph Method”, where mining occurs adjacent to a backfilled stope. Open stoping under consolidated backfill poses additional risk in terms of back collapse, and in turn dilution, and subsequent delays to the “mining cycle”, which includes drilling, blasting, mucking, and so on.

The design methodologies of sill mats are reviewed in the context of open stope “non-entry” mining at the Musselwhite mine. Deficiencies are addressed and a unique method of design is introduced via numerical and analytical simulations. Musselwhite and other underground operations are discussed in detail.
1.4 Thesis Overview

Chapter 2 details the mining method currently in use at the Musselwhite mine. Chapter 3 reviews existing literature detailing the types and uses of backfill, and backfill's intrinsic properties. This chapter also addresses existing analytical equations, and introduces two new analytical equations, to estimate rockfill loads and sill mat stability. A brief description of the software used in this study is given. Chapter 4 comprises all of the information gathered from the Musselwhite mine, including stress, structures, fabric analysis, cavity monitoring system surveys, geological sections, and others. Chapter 5 introduces relevant cemented rockfill guidelines by analyzing the stability of sill mats and the variables controlling stability, such as the vertical load exerted by unconsolidated backfill material. Chapter 6 further expands our understanding of the stability of open stopes in those cases where mining occurs adjacent to backfill material, by analyzing the effects of backfill on exposed wall dimensions. Chapter 7 provides a summary of the theoretical, empirical, and practical considerations of this thesis in concise, conclusive form. Chapter 8 proposes on-site instrumentation to evaluate the actual, practical effects of backfill material, and suggests possibilities for future work.

1.5 Contributions Made by Thesis

The principal contributions to the field of study are: a more efficient and cost effective design of sill mats within a mine situation; and a new method of evaluating the effect of backfill on open stope design. These are unique to the present state of knowledge for open stope operations.

Replacing an ore sill pillar with a cemented rockfill sill mat pillar will yield a higher overall recovery, i.e., improve the economics of the mining operation.

Where mining encounters a highly stressed back (e.g., the production drift ceiling) and/or weak rock mass, safety becomes a critical issue. In this case, the replacement of an existing ore pillar with a consolidated backfill pillar can improve the overall safety of the mine.

The methodology proposed in this thesis for the design of open stopes can be used to effectively reduce dilution as it exists in current mine designs, by accounting for the effect of backfill on wall stability. A technique based upon past practice and observed measurements,
coupled with numerical modeling, will be employed in the thesis to arrive at a new understanding of how backfill affects overall wall stability.

The models developed in this thesis attempt to reproduce as accurately as possible the behavior of the rock mass at the Musselwhite mine, i.e., stress regime, rock properties (e.g., strength), and so on. Therefore, these models relate to the Musselwhite mine operation itself, and should be considered applicable primarily to this specific case.

Although the models are intended to reflect accurately the reality of a particular mining situation, these models nevertheless require formal validation, i.e., calibration, by the acquisition of actual mine data, accomplished via proper instrumentation, conducted at the mine.

Additionally, this real world data application will enhance the accuracy of these numerical models, as it is carried out in future research, thus progressively increasing the usefulness of the design curves developed in this thesis.
2 THE MUSSELWHITE MINE

2.1 Introduction

Placer Dome Inc.'s Musselwhite Mine is located 430 km northwest of Thunder Bay, Ontario (figure 2-1).

Figure 2-1: Location of Musselwhite mine

The first underground production stope blast was made in March of 1997. The mill tonnage today is 4200 tonnes daily, of which 4000 tonnes of muck are extracted from underground, and the remainder comes from open pit. Figure 2-2 portrays a long section of the mine's orebody plunging at 12 to 15 degrees.

Figure 2-2: Long section of Musselwhite orebody
2.2 Geology

2.2.1 Regional Geology

The Musselwhite property is located in the central region of the Weagamow, North Caribou Lake metavolcanic, metasedimentary greenstone belt. The property is located in an area where the package has been isoclinally folded into a series of northwesterly trending antiforms and synforms (Musselwhite Project Feasibility, Geology Report, 1995). These structures plunge 12 to 15 degrees to the northwest. Important lithological units, referenced in this thesis, are:

- 4ea: Iron Formation, chert-grunerite-garnet-amphibole;
- 4f: Iron Formation, garnet-biotite schist;
- 4b: Iron Formation, chert-magnetite;
- A Vol: Volcanic, intermediate-mafic; and
- B Vol: Volcanic, intermediate-felsic.

Figure 2-3 portrays a geological cross-section of the T-Antiform in the Musselwhite gold deposit along grid line 10050 North. The cross-section depicts the distribution of gold bearing zones within the mixed basaltic and northern Iron Formation sequences.

![Figure 2-3: Geological cross-section through the T-Antiform Northern Iron Formation](image)
2.2.2 Mine Geology

This study deals principally with the gold mineralized zone in the T-Antiform. The gold mineralization is found predominantly within the sediment rich iron foundation called the “4ea”. More specifically, the gold is located in four subsidiary fold closures within the 4ea lithology, named the Wa, T, C, and S zones (figure 2-4). The Wa, T, and C zones are approximately located on the westerly limb of the antiform, and the S zone is located on the faulted, eastern limb.

![Geological cross-section showing the Wa, T, C and S zones](image)

Figure 2-4: Geological cross-section showing the Wa, T, C and S zones

In figure 2-4, the Wa, T, and C zones can be generally described as sub-parallel, steeply dipping, planar bodies of varying thickness. The Wa zone is the lowest and most westward zone on the fold limb, the T zone is in the middle, and C zone is at the top of the antiform.

These three ore zones are typically found adjacent to each other, but frequently have several meters of less significant mineralized 4ea between them. The zones vary in thickness from 0.5 meters to 25 meters, and typically represent a sectional height of 200 meters from the bottom of the Wa zone to the top of the C zone. The three zones lay in close proximity to each other.
For example, the Wa and T zone widths vary from 0 meters to 17 meters of waste materials (and typically, widths fall between 0 meters and 6 meters). The T and C zone widths vary from 0 meters to 13 meters of waste materials (and typically, widths fall between 0 meters and 9 meters). The S zone is a steeply dipping body positioned on the eastern limb of the anticline to the south, and running up to straddle both sides of the fold to the north (depicted in figure 2-5). The S zone is comprised of a short (45 meters), thick (10 meters) body in the south, and a thinner (6 meters), tall (150 meters), steeply dipping body to the north.
2.3 Underground Mining Methods

2.3.1 Avoca

Two underground longhole mining methods are currently employed at the Musselwhite mine: Avoca, or longitudinal retreat open stoping, with delayed backfill; and transverse open stoping (Musselwhite Project Feasibility, Mining Report, 1996). Longhole open stoping is the principal method employed at Musselwhite, because of its safety, productivity, and economic characteristics.

Where the ore is between 4 meters and 12 meters in width, Avoca, or longitudinal retreat is used. This method employs conventional longhole benching where the stopes are backfilled with waste rock composed of development waste and crushed material from open pit stripping.

Stopes are mined upwards, where the ore is mucked on top of the previously backfilled stopes below. The broken ore is usually remotely mucked on the lower sill. Rockfill material is dumped, using either scoops or trucks, from the top sill, filling the stope to just short of where the hanging wall exposure would exceed its predicted maximum stable span. The Avoca method demands periodic re-deposit of backfill to prevent the stope walls from collapsing into the open stope. The mine operation comprises sequentially: blasting, mucking, and backfilling. Slot raises are drilled and blasted at the end of each stope, to create the free face necessary for subsequent production blastholes to be fired.

![Figure 2-5: Longitudinal retreat open stoping -- Mining adjacent to backfill](image)

Drilling Equipment

Truck backfills after most ore is mucked

Retreating

Ore

Blasted Ore

Backfill

Floor can be of any type: Ore, backfill or sill (mat) pillar

Figure 2-5: Longitudinal retreat open stoping -- Mining adjacent to backfill
2.3.2 Transverse Retreat Open Stopping

Where the ore width exceeds 12 meters, it is typically mined transversely within 50 meter or 25 meter high stope blocks. The transverse stopes are backfilled with cemented and uncedmented rockfill. Cemented rockfill is used in primary stoping blocks and uncedmented rockfill is used in secondary stopes. Primary stope rockfill is crushed and screened to less than 5 inches, yielding a workable size that achieves the strength needed when mixed with cement binders. The material is dropped down a raise fill to the underground batching plant. Next, waste material is discharged through a chute into the truck box, and cemented slurry is then added via spray bars. Slot raises are drilled and blasted at each stope, creating new faces for each primary and secondary stope.

![Diagram of Transverse open stoping](image.png)

2.3.3 Drilling and Blasting Methods

The limits of the orebody define whether parallel or fanned 10 centimeter (4 inch) blastholes are drilled. These blastholes can be drilled as either down holes or up holes, depending on the location of the stope and its sequence.

Historically, ground control problems associated with drilling and blasting have been: inaccurate drilling; faulty timing of the holes; and/or discovery of unfamiliar geological structures. These are all important factors in wall stability.
2.3.4 Ore Handling

Ore is handled underground, by remote-controlled LHDs with 40-tonne-capacity articulated low profile trucks. They are used to haul the ore to the surface or to feed the bin of an underground crusher, 460 meters beneath the surface. Remote-controlled LHDs enhance both safety and productivity measures. Remotely, the operator controls the fully-functional LHD, ensuring maximum safety.

2.3.5 Ground Support

Consistent with Placer Dome’s support guidelines, pattern bolting sufficient to prevent small scale local failure, is used in all cases (Geotechnical Assessment of the Musselwhite Project, Golder Associates, 1996). The following table summarizes the type and pattern of development support employed:

Table 2-1: Support strength properties

<table>
<thead>
<tr>
<th>ROCK BOLT PROPERTIES</th>
<th>Bolt Strength</th>
<th>Yield Strength</th>
<th>Breaking Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(tonnes)</td>
<td>(tonnes)</td>
<td></td>
</tr>
<tr>
<td>5/8&quot; mechanical</td>
<td>6.1</td>
<td>10.2 (Grade 690MPa)</td>
<td></td>
</tr>
<tr>
<td>Split Set (SS-33)</td>
<td>8.5</td>
<td>10.6</td>
<td></td>
</tr>
<tr>
<td>Standard Swellex</td>
<td>N/A</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td>Yielding Swellex</td>
<td>N/A</td>
<td>9.5</td>
<td></td>
</tr>
<tr>
<td>Super Swellex</td>
<td>N/A</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>20mm rebar (#6)</td>
<td>12.4</td>
<td>18.5</td>
<td></td>
</tr>
<tr>
<td>25mm rebar (#8)</td>
<td>20.5</td>
<td>30.8</td>
<td></td>
</tr>
<tr>
<td>#6 Dywidag</td>
<td>11.9</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>#7 Dywidag</td>
<td>16.3</td>
<td>24.5</td>
<td></td>
</tr>
<tr>
<td>#8 Dywidag</td>
<td>21.5</td>
<td>32.3</td>
<td></td>
</tr>
<tr>
<td>#9 Dywidag</td>
<td>27.2</td>
<td>40.9</td>
<td></td>
</tr>
<tr>
<td>#10 Dywidag</td>
<td>34.6</td>
<td>52</td>
<td></td>
</tr>
<tr>
<td>1/2&quot; Cable bolt</td>
<td>15.9</td>
<td>18.8</td>
<td></td>
</tr>
<tr>
<td>5/8&quot; Cable bolt</td>
<td>21.6</td>
<td>25.5</td>
<td></td>
</tr>
</tbody>
</table>

#6 refers to 6/8", #7 refers to 7/8", #8 refers to 8/8" diam etc.

<table>
<thead>
<tr>
<th>BOND STRENGTH</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Split Set</td>
<td>0.75 – 1.5 tons/ft</td>
<td>0.7 – 1.4 tonnes/ft</td>
</tr>
<tr>
<td>Swellex</td>
<td>3 – 5 tons/ft</td>
<td>2.7 – 4.6 tonnes/ft</td>
</tr>
<tr>
<td>Cable Bolt (5/8&quot;)</td>
<td>29 tons – 3ft</td>
<td>10.2 (Grade 690MPa)</td>
</tr>
<tr>
<td>Rebar 18 tonne</td>
<td>20 tons – 12&quot;</td>
<td>18 tonnes – 12&quot; (granite)</td>
</tr>
</tbody>
</table>
Chapter 2 – The Musselwhite Mine

<table>
<thead>
<tr>
<th>SCREEN – BAG STRENGTH 4ft X 4ft PATTERN</th>
</tr>
</thead>
<tbody>
<tr>
<td>4’ X 4” Welded wire mesh (4 gauge)</td>
</tr>
<tr>
<td>4’ X 4” Welded wire mesh (6 gauge)</td>
</tr>
<tr>
<td>4’ X 4” Welded wire mesh (9 gauge)</td>
</tr>
<tr>
<td>4’ X 2” Welded wire mesh (12 gauge)</td>
</tr>
<tr>
<td>2” chainlink – 11 gauge bare metal</td>
</tr>
<tr>
<td>2” chainlink – 11 gauge galvanized</td>
</tr>
<tr>
<td>2” chainlink – 9 gauge bare metal</td>
</tr>
<tr>
<td>2” chainlink – 9 gauge galvanized</td>
</tr>
</tbody>
</table>

4 gauge = 0.23” diam, 6 gauge = 0.20”, 9 gauge = 0.16”
11 gauge = 0.125”, 12 gauge = 0.11” diam

shotcrete shear strength = 2MPa = 200 tonnes/m²

2.3.5.1 Development Support

Placer Dome’s support guidelines indicate that pattern bolting sufficient to prevent small scale local failure be used in all cases. The type of support that should be employed depending on the span of the opening, is summarized in the following table:

Table 2-2: Development support

<table>
<thead>
<tr>
<th>Span</th>
<th>#6 rebar 1.4 x 1.4 m</th>
<th>#8 rebar 1.8 x 1.8 m</th>
<th>Fully Grouted</th>
<th>Face Plate</th>
<th>Cable Bolts 3.6 x 3.6 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 5.5m</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>5.5m – 7.2m</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>7.2m &lt;</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
</tbody>
</table>

2.3.5.2 Stope support – Stope backs and walls

It is standard practice in Canada to use support in all working areas. The accepted convention is that the length of the support should not be less than one-third of the minimum span of the supported excavation. Assuming that rock bolts with a maximum length of 2.4
meters are used, cables should be employed in all excavations where the minimum span exceeds 7.2 meters.

Cables effectively support hanging walls and foot walls, and prevent cave-in, which would affect production and increase dilution if not addressed sufficiently.

2.3.6 Backfill

The Musselwhite orebody is of variable width and dip. Accordingly, there are a number of different stope layouts. The two main layouts are longitudinal retreat, where the fill exposure is of limited span and up to 30 meters high, and transverse stoping where the fill spans up to 30 meters with heights of up to 25 or 50 meters.

Two backfill stations are located at the 150 mL and the 200 mL. Given that the bulk of the mine production today comes from the 400 mL and below, backfilling has become a time consuming and costly task.

Open pit, screened, cemented rockfill is used for backfilling transverse primary stopes. As waste is mined, selected rock is stockpiled for crushing. Crushed rock is dumped into a fill raise and then transferred to trucks via a chute at the bottom of the raise. Cement is transferred via a borehole to an underground cement slurry station, or batch plant, located adjacent to the fill raise, where the cement is mixed with rockfill. Spray bars are used to coat the rockfill with a pre-measured amount of cement slurry, in accordance with relevant strength requirements. Trucks haul the fill to the stope and dump the fill at the top of the stope.

The system is comprised of several components:

- crusher;
- on-site cement storage (cement and fly ash silos);
- boreholes for cement transfer underground;
- raises for transfer of rockfill underground; and
- underground fill stations (including the slurry plant).

Generally, the upper bound for particle size delivered to the top of the fill raise is set within a range of 150 mm to 200 mm. Waste rock consists typically of meta-volcanic rock with estimated uniaxial strengths in excess of 150 MPa, which is ideal for CRF composition.
2.3.6.1 Mining Underneath Backfill – CRF Sill Mats

A sill mat pillar is a cemented, enriched, bottom floor of a backfilled stope. It is used in place of an ore sill pillar, maintaining stability when mining the stope below. Musselwhite’s sill mats are constructed from several batches of screened rockfill material, recovered from run of mine development waste and/or open pit crushed waste rock (crushed to < 12.5 centimeters). Binder is mixed at the surface plant. The resultant slurry is transported below ground via boreholes. Each truck-load is coated with the binder mixture using spray bars, transported and end-dumped onto the floor in a retreating fashion. A scoop (LHD) is used to bring the sill mat to proper elevation, as required.

To date, 3 sill mats have been constructed: the 275 mL, the 300 mL, and the 375 mL, portrayed in figure 2-7.

![Figure 2-7: Cemented rockfill sill mats constructed: 275, 300 and 375 m Levels](image)

2.3.6.2 Mining Adjacent to Backfill

Stopes are mined upwards, using sub-level open stoping methods. The ore is mucked on the floor of backfill material and deposited on the level below. The broken ore is normally remotely mucked on the bottom sill. Rockfill material is dumped, using either scoops or
trucks, from the top sill to the point just before the hanging wall exposure would exceed its predicted maximum stable span.

The longitudinal retreat and transverse open stoping methods both require periodic placement of backfill to support the stope walls.

This mine operation consists sequentially of: blasting; mucking; and backfilling. Slot raises are drilled and blasted at the end of each stope to create the free face needed for subsequent production blastholes. Remote scoops are utilized to muck ore from the blasted stopes. Use of remote-control mucking improves both safety and productivity.

Rockfill used for filling mined stopes comes from underground waste development, or from open pit rock material. Open pit rockfill material is crushed and screened to a maximum size of 10 cm, providing crushed rock dimensions consistent with strength guidelines for mixing with cement (< 5% for primary stopes; < 8-10% for sill mat pillars). The material is dropped down a raise fill to the underground batching plant, where it is discharged through a chute and onto a truck or scoop. Cemented slurry is then applied with spray bars to meet shear and tensile strength requirements.

Maintaining a high quality backfill product is crucial to the safety and efficiency of the mining operation, while maximizing ore recovery, and leaving behind minimal ore pillars.
2.4 Equipment

Mobile equipment in use is summarized by table 2-3.

Table 2-3: Mobile Equipment List

<table>
<thead>
<tr>
<th>Type of Equipment</th>
<th>Make/Model</th>
<th>Number of Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drilling Equipment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Production drills</td>
<td>Tamrock Solo 1000 Sixty</td>
<td>2</td>
</tr>
<tr>
<td>Development drills</td>
<td>Tamrock Minimatic H205D</td>
<td>3</td>
</tr>
<tr>
<td>Bolters</td>
<td>Tamrock Robolt H320-30C</td>
<td>3</td>
</tr>
<tr>
<td>Miscellaneous drills</td>
<td>MacLean Blockholer</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Boart BCI-2</td>
<td>1</td>
</tr>
<tr>
<td>Production Scoops</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 yd³</td>
<td>Tamrock EJC-130</td>
<td>1</td>
</tr>
<tr>
<td>8 yd³</td>
<td>Tamrock Toro T500D</td>
<td>3</td>
</tr>
<tr>
<td>9 yd³</td>
<td>Tamrock Toro T650D</td>
<td>5</td>
</tr>
<tr>
<td>11 yd³</td>
<td>Tamrock Toro 0011</td>
<td>1</td>
</tr>
<tr>
<td>Production Trucks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30 tonne</td>
<td>Tamrock EJC 430</td>
<td>1</td>
</tr>
<tr>
<td>40 tonne</td>
<td>Tamrock Toro 40D</td>
<td>9</td>
</tr>
<tr>
<td>Ancillary Equipment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Explosives loader</td>
<td>Tamrock ALB45</td>
<td>2</td>
</tr>
<tr>
<td>Scissor lift</td>
<td>Teledyne SL6-812</td>
<td>3</td>
</tr>
<tr>
<td>Boom truck</td>
<td>Teledyne</td>
<td>1</td>
</tr>
<tr>
<td>Grader</td>
<td>Caterpillar M-120</td>
<td>1</td>
</tr>
<tr>
<td>Personnel vehicle</td>
<td>Toyota Landcruiser</td>
<td>17</td>
</tr>
<tr>
<td>Bulk explosives truck</td>
<td>ICI U-101-1</td>
<td>1</td>
</tr>
</tbody>
</table>

This chapter reviewed Musselwhite’s mining methods, types of support, handling of ore and waste materials, and types of equipment in current use. The next chapter will discuss design methodologies in current use.
3 REVIEW OF DESIGN METHODOLOGIES

This review of pertinent, existing literature is sub-divided into the following topics:

- uses, types, and properties of backfill;
- vertical load exerted by rockfill;
- analytical sill mat design;
- empirical stability graph methods; and
- software utilized in the analysis.

3.1 Backfill

Backfill is an increasingly important component of underground mining operations around the world. Following is a brief overview of current methods of backfill technology, focusing on rockfill. Rockfill is the principal backfill material used at the Musselwhite operation.

Disposal of backfill underground not only reduces the environmental impact and footprint, but also provides the basis of an engineering material that can be used to improve both the ground conditions, and the economics of mining. Carefully engineered and efficiently operated backfill systems can significantly enhance the productivity of a given mining operation. On the other hand, badly engineered backfill system can become a serious impediment to mining operations and, in the worst case scenario, compromise safety.

Whether composed of tailings, alluvial sand, or coarse rockfill, backfill can be treated as a special form of soil. Therefore, several of soil’s mechanical properties and relationships can be applied to backfill. Note that mechanical and cure properties of fill deposited underground may vary considerably, depending upon the fill material’s intrinsic properties, the method of preparation, and the location and condition of the mine environment.

3.1.1 Purpose of Backfill

Backfill has multiple purposes, as indicated in figure 3-1. The primary purposes are to improve hangingwall stability, and to permit increased ore extraction. Other important application purposes include dilution control and regional support. Although much less
common, some interesting applications might include ventilation improvement, fire control, and environment protection (De Souza, et al., 1998).

![Figure 3-1: Purposes of backfilling](image)

For wall stability using backfill, the objective is not to transmit rock stresses, but to reduce the uncontrollable convergence of rock mass so that the rock itself will retain its load carrying capacity, and in turn, decrease cracking propagation of crown pillars and abutments (Barrett et al., 1978). This also helps to prevent cave-ins and roof falls, minimize surface subsidence, and enhance pillar recovery (Coates, 1981). The ultimate result is less deterioration of ground conditions in the mine, and thus operation and safety are improved overall.

Adding small percentages of ordinary Portland cement provides cohesive strength, and self-support when exposed in vertical faces to adjacent pillar mining, as well as to the mining conducted below. The self-supporting nature of backfill permits higher recovery of pillars. This, in turn, allows greater exploitation of the mining reserves, and improves the economics of the mining operation. Increased ore recovery results in a longer mine life.
In some mining methods, backfill forms a working platform for mine personnel and equipment. Backfill must be capable of supporting this traffic. Cement is usually not required in such an application.

Depositing backfill underground directly reduces the quantity of waste to be disposed of on the surface. This results in direct operating and capital cost benefits, and reduces future rehabilitation costs.

Nantel (1998) initiated a trend now common in Canada, where environmental authorizations for future underground mines require the return of the maximum available quantity of mine wastes to the underground operation. This trend has reached its logical limit, as seen in the recent Australian government recommendation to approve a project for the proposed Jabiluka uranium mine, which is encircled by the world famous Kakadu National Park (inscribed on the World Heritage List). For this project, a maximum percentage of milling wastes are required to be re-deposited underground.

Research by Yu (unpublished) and Stone (1993) provides a good background reference source regarding the placement of cemented rockfill, typical strength characteristics and its applications.

3.1.2 Types of Backfill

Hassani and Archibald (1998) stated that backfill refers to any waste materials re-deposited into voids mined either for disposal or some other engineering task. These materials include waste development rock, deslimed and whole mill tailings, quarried and crushed aggregate and alluvial or aeolian sands. Other exotic backfill components may include ice and salt. The waste materials are often combined with cement or other pozzolanic (volcanic ash) binders to improve their strength properties.

The three most common backfill types are:

- **Hydraulic backfill** Deslimed mill tailings slurry, with densities up to 70%cw (i.e., concentration of solids by weight) -- the coarser fractions are deposited underground as hydraulic backfill and the slimes rejected to the surface dam;
Chapter 3 – Review of Design Methodologies

3.1.2.1 Rockfill

Rockfill can be grouped into unconsolidated and consolidated types of material. The latter comprises binding mixtures that achieve the required strength for a given purpose.

Consolidated, or cemented, rockfill (CRF) includes classified or unclassified aggregate, mixed with a variety of types and quantities of binder materials. Typically, unconsolidated, or uncemented, rockfill (URF) is pre-mixed with cement binder, usually in slurry form, prior to its entry into the stope, forming a reasonably homogenous material at that time. Consolidated rockfill is commonly employed in large exposures where undermining can occur, future exposure of fill walls is expected, and significant resistance to wall movement is desired. The rockfill stiffness so achieved produces significant advantages to operations seeking ground control at sites subject to high ground stresses.

URF also facilitates an inexpensive method of disposing of development waste or surface open pit waste.

3.1.2.2 Uncemented Rockfill – URF

URF is waste material deposited without additives. Although URF has a limited ground support capability because of its minimal resistance to closure, URF will prevent wall slough. The free standing height capacity of such fill is negligible. URF is used when the filled stope won’t be exposed to future pillar recovery operations. In this case, stopes surrounding the filled stope will have been mined and filled before pillar recovery. URF is generally used for void filling and to provide some measure of passive wall support to resist localized ground
movement. It is a relatively low cost fill medium, which is both expedient and relatively easy to place in situ.

### 3.1.2.3 Cemented Rockfill with Portland cement Slurry – CRF

A variation on cemented rock backfill widely used in Canada makes use of crushed waste rock and Portland cement slurry aggregated at rates of approximately 5% cement by weight. Consequently, the overall sizing curve is low in fines content (i.e., size distribution of rockfill). This reduces the workability of the deposited backfill and demands higher cement aggregation to provide equivalent strength performance. The technique is currently in use at the Musselwhite mine, where a surface batching plant produces slurry, which is then delivered via boreholes to an underground mixing plant.

Note that CRF yields a higher strength fill, demanding lower quantities of cementing agents in comparison to cemented hydraulic fill materials (including pastefill). Aggregated with equivalent binder components, CRF exhibits uniaxial compressive strengths that can be two to three times higher than consolidated hydraulic fill (including pastefill). CRF rockfills also exhibit higher moduli of elasticity, cohesion, and friction angle characteristics than hydraulic fill mixtures (including pastefill) composed of similar cement contents.

### 3.1.3 Backfill Properties

Rockfill particles are in constant contact with each other, i.e., particles do not collide but rather roll, rub, and scrape against one another. This type of stress is called frictional stress, as opposed to collisional or kinetic stress, which occurs in more diluted solutions.

When granular material does not move or flow to any extent, the only force acting on the grains is gravity. Basic engineering theory states that for any fluid, the total stress at the bottom of a granular pile of height $h$ would be:

$$P = \rho \cdot g \cdot h \quad \text{3-1}$$

Where:
- $g$ is the acceleration of gravity; and
- $\rho$ is the bulk density of the rockfill material.
Theoretically, the total stress at the bottom of the waste fill is a normal stress along the vertical direction \( (h) \), and therefore is an isostatic pressure, solely resulting from the weight of the rockfill pile. In practice this is not true. In fact, beyond a sufficient height of the rockfill column, the pressure reaches a maximum value and will not increase further, regardless of the height of the fill. This occurs because rockfill supports frictional shear stress, even in a static situation. Additionally, if side walls exist within the stope, they can support the extra weight of the rockfill column. And so, the total stress at the bottom of the rockfill pile is a combination of normal stress and shear stress.

In order to predict the ground support capacity of a backfill system, it is essential to first define, and to understand, the significant properties of the fill material. Since most fill material can be defined as either granular or fine-grained soils, evaluation of any fill property involves the principles of soil mechanics. However, soil mechanics alone cannot describe all of the properties of cemented fill. Aggregation of cement and other pozzolans to a fill, transforms the fill from natural material, e.g., soil, to an engineered material, making the process of characterizing fill properties more complex. Extensive evaluation of the effects of cement aggregation is therefore necessary.

Existing detailed studies and laboratory work examine cemented fills and the properties significant in their design. Laboratory testing indicates that the relevant properties of cemented fill (Knissel and Helms, 1983) are:

- strength (i.e., uniaxial compressive strength);
- deformation behavior;
- cohesion and angle of internal friction;
- density and porosity; and
- consistency of the mixture.

As with uncemented fill, strength is the principle concern in cemented fills. Certain properties of the fill material including friction angle, density, and porosity, have the same effect on cemented fill strength as on uncemented fill strength.

Cement aggregation improves the strength properties of backfill material. The most evident increase in strength of a cemented fill over an uncemented fill is in shear strength.
Cement bonds formed between fill particles produce a cohesive component in the fill's shear strength, absent in an uncedmented fill.
The relationship between shear stress and normal stress is commonly referred to as the Mohr-Coulomb relationship for frictional shear stress.

In its basic form, it is:

$$\tau = c + \sigma_n \cdot \tan \phi$$  \hspace{1cm} (3-2)

Where:
- \(\tau\) is the fictional shear stress (MPa, psi, etc.);
- \(\sigma_n\) is the normal frictional stress (MPa, psi, etc.); and
- \(c\) is the cohesion of the material (MPa, psi, etc.).

Cohesion, \(c\), is a known material property (describing the cohesive state of grains) normally negligible in uncemented fill material, and \(\phi\) is the angle of repose (i.e., the angle of internal friction of the material).

The following figure describes equation 3-2:

![Frictional Stress: Mohr-Coulomb Plane](image)

**Figure 3-2: Mohr-Coulomb failure envelope for a cohesive fill material**

At yield, the higher the frictional stress, the higher the shear stress. The more interconnected the particles, i.e., the more cohesive, and/or the higher the angle of friction, the higher the shear stress, as indicated in figure 3-2.
The Mohr-Coulomb law described by equation 3-2 and portrayed in figure 3-2, is a yielding law asserting that a material will yield by shearing on a surface element if $\tau$ attains a critical value defined by equation 3-2. This linear relationship is sometimes called the “yield line”. Below the yield line, the material response will be rigid or elastic and will not typically undergo strain, and if it does it is merely elastic strain. Elastic strain is negligible in uncedmented rockfill material.

Figure 3-3 portrays the Mohr-Coulomb envelope for a cohesionless fill material. If the shear stress is increased for a given normal stress so that the stress state of the material remains at yield, then plastic strain, i.e., yielding, will result. A state of stress cannot exist above the Mohr-Coulomb yield line. When yield stress is reached, particles will simply slide over one another.

![Frictional Stress: Mohr-Coulomb Plane](image)

\[ \tau = \sigma n \tan(\phi) \]

\( \phi = \text{Friction Angle} \)

**Figure 3-3: Mohr-Coulomb failure envelope for a cohesionless fill material**

In this case, shear stress of the rockfill material is due solely to the frictional component and is mobilized by the normal stress.
The angle of repose (or angle of internal friction) is evident in the following figure:

![Figure 3-4: Angle of repose $\phi$ for cohesionless fill material](image)

The angle of repose, portrayed in figure 3-4, is low when grains are smooth, coarse, or rounded, and high for sticky, sharp, irregular, or very fine particles. Typically, it is between 15 and 50 degrees. Musselwhite rockfill friction angles range from 35 to 40 degrees.

### 3.1.4 Parameters Affecting Cemented Rockfill Strength

The following parameters are considered essential to final rockfill strength (Kuganathan, et al., 2001):

- maximum size of the aggregate;
- grading of the aggregate;
- binder content of the mix; and
- water content of the mix.

Suitable grading of the aggregate is important, not only to reduce porosity, i.e., the void ratio of the fill, but also to increase the number of rock to rock contacts per unit area. The strength of the fill depends on the number of cement bonds that develop at these contact points. The strength and quality of CRF depends on how well the rockfill particles are coated with the cement slurry before they are deposited in the stope. In a well controlled operation,
the resultant fill will exclude uncemented rockfill pockets, and a nearly homogeneous CRF fill mass is thus achievable.

In the majority of underground mining operations reviewed, sized rockfill aggregate is mixed with cement slurry, typically at a cement content of 5 to 6 percent, by weight (cw), and a pulp density of 50 to 60% (cw). This type of fill may exert active pressure on contact with stope walls, providing not only ground support but also improvement in the inherent wall rock strength. In the case of consolidated rockfill, there normally occurs no drainage problem, and a high fill quality can often be achieved if the materials are mixed properly. With these rockfill materials however, segregation control can be difficult, and quality control may be variable. In this regard, Stone (1993) presents a methodology for the optimization of mix designs for cemented rockfill, and suggests laboratory evaluations for verification of the design parameters. Furthermore, a program of quality control testing and performance monitoring is outlined to improve the reliability of the design.

Segregation of consolidated rockfill is often unavoidable but can be minimized when fill operations are well planned and closely monitored.
3.2 Vertical Load on Cemented Rockfill Sill Mat Pillars

3.2.1 Sill Mat Pillar Definition

Sill pillars are ore blocks left between working levels in an underground mine to support the overlying mine backfill, during removal of the underlying ore. Steeply dipping ore bodies should be mined bottom up but for economic reasons and/or because of stability constraints (i.e., underhand cut and fill), upper ore levels are usually mined first. Sill pillars are commonly used in steeply dipping ore zones of limited width.

A typical situation is portrayed in figure 3-5:

![Figure 3-5: Sill mat pillar cross-section](image)

Accordingly, mineable values in the ore generally make it economically feasible to mine the sill pillar. It is necessary to create an artificial sill, which will support the overlying unconsolidated fill, after the stope below is mined. Such sill mats can be constructed from either cemented rockfill or hydraulic and paste backfill material.

In analyzing sill mat stability, the strength of backfill necessary to maintain stability is a function of the stresses generated within and around the sill mat and the fill mass.

These stresses are caused by:
- self-weight of the sill mat;
- vertical load of unconsolidated backfill;
- blast damage/abrasion; and
- ground movements.
The most significant design loading used in this thesis is the weight of the fill material. The stresses generated by this weight are a function of fill density, fill friction angle, height, and span.

A number of methods have been proposed for analyzing the stability of cemented sill mats. These generally use free standing wall or two-dimensional stope stability concepts. Mitchell's (1981) approach is reviewed in this chapter.

3.2.2 Background – Loads Acting by Unconsolidated Material

When backfill material is dumped onto a horizontal surface, it heaps into a volume conical in shape, forming a specific angle $\phi$ with the horizontal plane. This angle is characteristic of each type of material and is called the natural angle of repose of the material. This is the angle of internal friction. If the material is dumped into a confined space such as that of a stope, it will exert pressure on the walls and on the floor of the stope. The resultant thrust, due to the friction of the material on the walls, is oblique in relation to the surface of the walls. This thrust has two components, one, N, normal to the wall, and the other, T, parallel to the wall (Reimbert, 1976).

The normal pressure is also called lateral thrust (figure 3-6):

![Diagram](image)

Figure 3-6: Natural angle of repose of the material (left) and the oblique, normal and tangential components of the force (right)
The material’s angle of friction on the walls is termed $\phi'$, and the corresponding coefficient of friction is $\tan \phi'$. As a function of the oblique thrust $Q$ defined above, the two components $N$ and $T$ are:

\[ N = Q \cdot \cos \phi' \quad 3-3 \]
\[ T = Q \cdot \sin \phi' \quad 3-4 \]

Therefore:

\[ T = N \cdot \tan \phi' \quad 3-5 \]

Where $T$ is the load balanced by the friction corresponding to the thrust $N$.

At a given depth of the stope, the load, i.e., total vertical pressure, is the difference between the total weight of the backfill material and the total load offset by the friction of the material on the walls.

*In situ* measurements tell us that pressure increases with depth. However, because of the friction of the material on the walls, the pressure on the stope floor is merely a fraction of the weight of the confined material. Furthermore, at great depths the pressure reaches a constant maximum value (Reimbert, 1976). The curve representing pressure (plotting depth on the $x$-axis, and pressure on the $y$-axis) is asymptotic parallel to the $x$-axis, in correspondence with the maximum vertical pressure (portrayed in figure 3-7).
Vertical Stress at Depth
for Confined Material

Maximum Vertical Pressure

Figure 3-7: Vertical stress at increasing depth for a confined material

3.2.3 Maximum Horizontal Pressure on the Walls of the Stope

A behavior similar to that portrayed by figure 3-7, can be inferred from the curve representing the lateral thrust on the walls. This curve is asymptotic parallel to the x-axis where the y-axis plots maximum thrust.

The value of the maximum horizontal thrust can be determined by the following method (figure 3-8):

Figure 3-8: Pressure on a horizontal slice of thickness $\delta z$
The weight of a horizontal slice of thickness $dz$ in the interior of the stope is:

$$S \cdot dz \cdot \gamma$$ \hspace{1cm} \text{(3-6)}

Where:

- $S$ = internal area of the cross-section of the stope;
- $\gamma$ = unit weight of the backfill material; and
- $dz$ = thickness of the horizontal slice.

The weight exerts a lateral pressure on the walls over the entire perimeter:

$$\sigma_x(z) \cdot P \cdot dz$$ \hspace{1cm} \text{(3-7)}

Where:

- $\sigma_x(z)$ = lateral thrust at a depth $z$; and
- $P$ = internal perimeter of the stope.

Pressure increases with depth up to the maximum $\sigma_{x_{\text{Max}}}$, at which point the friction on the walls balances the actual weight of the slice considered. Therefore:

$$\sigma_{x_{\text{Max}}} \cdot P \cdot dz \cdot \tan \phi' = S \cdot dz \cdot \gamma$$ \hspace{1cm} \text{(3-8)}

From which the value of the maximum thrust can be determined:

$$\sigma_{x_{\text{Max}}} = \frac{\gamma \cdot S}{P \cdot \tan \phi'}$$ \hspace{1cm} \text{(3-9)}

Note that, for stopes of sufficient strike length (i.e., in theory, infinite), the hydraulic radius $S/P$ tends toward one half the span of the stope (see equation 3-9). Equation 3-10 indicates the maximum lateral thrust within a stope of sufficiently long strike:

$$\sigma_{xx_{\text{Max}}} = \frac{\gamma \cdot L}{2 \cdot \tan \phi'}$$ \hspace{1cm} \text{(3-10)}

Where:
3.2.4 Coefficient of Lateral Earth Pressure

Estimating lateral earth pressures is not a trivial problem. Nor is it a new problem. Couplet and Coulomb developed their theories in 1726 and 1776 (respectively), and Rankine developed his theory in 1857. Flexural movement of the structure must occur for the predictions of these theories to be valid, and whether it is translation/rotation into, or out of, the soil mass in question, active (or passive) lateral earth pressure will develop.

The coefficient of lateral earth pressure implies a constant relationship between the average horizontal, $\sigma_H$, and vertical, $\sigma_V$, stresses, independent of the geometry of the fill volume. While there is no physical reason for such a relationship to hold, in practice, routine industrial calculations indicate that the results implied by this relationship are reasonably accurate empirically.

Equation 3-11 portrays this relationship (the coefficient of lateral earth pressure):

$$ K = \frac{\sigma_H}{\sigma_V} $$

3.2.4.1 Categories of Lateral Earth Pressure

There are three categories of lateral earth pressure and each depends upon the movement experienced by the vertical wall upon which the pressure is acting.

These categories are:

- active earth pressure;
- passive earth pressure; and
- at-rest earth pressure.
Active pressure develops when the wall is free to move outward. E.g., a retaining wall where the soil mass may stretch sufficiently to mobilize its shear strength. On the other hand, if the wall moves into the soil, then the soil mass is compressed sufficiently to mobilize its shear strength where passive pressure develops.

This problem might occur along the section of the wall that is below grade, on the opposite wall from that of the higher section.

In order to develop full active pressure or full passive pressure, the wall must move. If the wall does not move sufficiently, full pressure will not develop. If full active pressure does not develop behind a wall, then the pressure will be higher than the expected active pressure. Significant movement is necessary to mobilize the full passive pressure. This is illustrated in figure 3-9.

Most earth retaining problems involve the movement of the structure away from the soil mass, known as the “active condition”. Terzaghi (1920) demonstrated the significance of lateral earth pressures developed on structures that experience no movement, and termed them “at-rest earth pressures”. Note the at-rest condition, portrayed by figure 3-9(b), where the wall rotation is equal to zero (i.e., the condition for zero lateral strain).

Figure 3-9 shows that:

- as the wall moves away from the soil backfill (see figure 3-9(c)), the active condition develops and the lateral pressure against the wall decreases with wall movement, until the minimum active earth pressure force is reached;
- as the wall moves toward (i.e., into) the soil backfill (see figure 3-9(a)), the passive condition develops and lateral pressure against the wall increases with wall movement until maximum passive earth pressure is reached – the intensity of the active/passive horizontal pressure, which is a function of the applicable earth pressure coefficient, depends on wall movement, as it affects the degree of shear strength mobilized in the surrounding soil; and
- at-rest pressure develops when the wall experiences no lateral movement, which typically occurs when the wall is restrained from movement, as in the case of a stope (see figure 3-9(b)).
3.2.4.2 Calculating Lateral Earth Pressure Coefficients

Lateral earth pressure is related to vertical earth pressure by coefficients, termed as follows:

- active earth pressure coefficient (Ka);
- passive earth pressure coefficient (Kp); and
- at-rest earth pressure coefficient (Ko).

Lateral earth pressure is equal to vertical earth pressure multiplied by the appropriate earth pressure coefficient.
Since soil backfill is typically granular material, such as crushed rock, sand, silty sand, sand with gravel, etc., the backfill material that exerts pressure on the wall can be treated as coarse-grained non-cohesive material.

### 3.2.4.2.1 Active and Passive Earth Pressure Coefficients

When discussing active and passive lateral earth pressure, there are two relatively straightforward classical theories (among others) in widespread use: Rankine earth pressure, and Coulomb earth pressure.

The Rankine theory assumes that:
- there is no adhesion or friction between the wall and soil;
- lateral pressure is limited to vertical walls;
- failure (in the backfill) occurs as a sliding wedge along an assumed failure plane defined by $\phi$;
- lateral pressure varies linearly with depth, and the resultant pressure is located one-third of the height above the base of the wall; and
- the resultant force is parallel to the backfill surface.

The Coulomb theory is similar to the Rankine theory, except that:
- there is friction between the wall and soil and accounts for this by using a soil-wall friction angle of $\delta$ ($\delta$ ranges from $\phi/2$ to $2\phi/3$ and a $\delta$ equal to $2\phi/3$ is commonly used);
- lateral pressure is not limited to vertical walls; and
- the resultant force is not necessarily parallel to the backfill surface because of the soil-wall friction value $\delta$.

The Rankine active and passive earth pressure coefficients for a horizontal backfill surface are calculated as follows:

$$(Active) \rightarrow K_a = \frac{1 - \sin(\phi)}{1 + \sin(\phi)} = \tan^2\left(45 - \frac{\phi}{2}\right)$$  \hspace{1cm} 3-12
(Passive) \( K_P = \frac{1 + \sin(\phi)}{1 - \sin(\phi)} = \tan^2(45 + \phi/2) \) \hspace{1cm} 3-13

The Coulomb active and passive earth pressure coefficients are more complex expressions that depend on the angle of the back wall, the soil-wall friction value, and the angle of the backfill. The Coulomb active and passive earth pressure coefficient for the specific case of a vertical back wall angle and horizontal backfill surface (not shown) yields results equivalent to the Rankine method (equations 3-12 and 3-13).

### 3.2.4.2.2 At-Rest Coefficient

Generally, at-rest earth pressure is the horizontal component of the in situ stress state, or the horizontal pressure acting on an earth retaining structure. At-rest lateral earth pressures can be shown to be some multiple of the vertical stress at any point, illustrated in the following ratio (Terzaghi, 1920):

\[
K_O = \frac{\sigma_h}{\sigma_v} \hspace{1cm} 3-14
\]

Where:
- \( \sigma_h \) is the horizontal pressure;
- \( \sigma_v \) is the vertical pressure; and
- \( K_O \) is the coefficient of earth pressure at-rest.

A more specific definition is offered by Bishop (1958): “The coefficient of earth pressure at-rest, is the ratio of the lateral to the vertical effective stresses in a soil consolidated under the condition of no lateral deformation, the stresses being principal stresses with no shear stress applied to the planes on which these stresses act” or:

\[
K_O = \frac{\sigma'_h}{\sigma'_v} \hspace{1cm} 3-15
\]

Where:
- \( \sigma'_h \) is the horizontal principal effective stress; and
\( \sigma'_v \) is the vertical principal effective stress.

Existing published relationships depend upon the soil’s engineering values to calculate the at-rest earth pressure coefficient. One common earth pressure coefficient for the at-rest condition used with granular soil is called the "Neutral Earth Pressure Method", derived by Tschebotarioff (1973):

\[
K_o = \frac{\nu}{1 - \nu} \tag{3-16}
\]

Where:

- \( \nu \) is the Poison’s ratio.

The assumption is that the force of gravity affects the elastic mass of material and therefore lateral movement is prevented.

An alternate solution for \( K_0 \) can be found via Jaky’s (1944) equation:

\[
K_o = 1 - \sin(\phi) \tag{3-17}
\]

Where:

- \( \phi \) is the soil friction angle value.

### 3.2.5 Maximum Vertical Load Exerted by Backfill on the Floor of the Stope

After determining the maximum horizontal pressure or lateral thrust \( \sigma_{x,\text{Max}} \), shown in equation 3-9, on the wall of the stope, the next step is to find the maximum vertical pressure exerted by the backfill. From the (previously defined) coefficient of lateral earth pressure \( K \), corresponding with the horizontal to vertical stress ratio, maximum vertical stress can be found using equation 3-18:

\[
\sigma_{y,\text{Max}} = \frac{\gamma \cdot S}{P \cdot K \cdot \tan(\phi)} \tag{3-18}
\]

Strike length of sufficient magnitude allows for the hydraulic radius to be considered as one-half of the stope span. The maximum vertical stress will be:
3.2.5.1 Terzaghi's Formulation

Another approach to estimating the vertical load exerted by backfill material is found in Terzaghi et. al. (1948). Terzaghi derives a solution to the first order linear differential equation:

\[
\frac{\partial \sigma_y}{\partial y} = \left( \gamma - \frac{2 \cdot c}{L} \right) - \left( \frac{2 \cdot K \cdot \sigma_y \cdot \tan \phi}{L} \right)
\]

Where:

- \( c \) is cemented rockfill cohesion.

For cohesionless soils, the solution to the first order differential equation yields the equation for maximum vertical stress:

\[
\sigma_{y, \text{Max}}(z) = \left( \frac{\gamma \cdot L}{2 \cdot K \cdot \tan(\phi)} \right)
\]

3.2.5.2 Blight's Formulation for Inclined Stopes

Blight (1984) proposed an approximate solution for the stresses in an inclined planar stope. This solution applied to the design of inclined rectangular stopes, produces the equation:

\[
\sigma_{y, \text{Max}} = \left( \frac{\gamma \cdot L}{2 \cdot K \cdot \tan(\phi)} \right) \cdot \sin(\beta)
\]

Where:

- \( \beta \) is the inclination of stope.
3.2.6 Vertical Load of Unconsolidated Material as a Function of the Material’s Height

Employing the maximum vertical load rather than the actual vertical load, dependent upon the height of the backfill, overestimates the strength required to withstand the load on a sill mat. This in turn leads to an increase in cost. For that reason the preferable application of vertical loads is as a function of the backfill height.

There are numerous formulations of silo theory, two of which are discussed here. Silo theory only applies to vertical silos and thus is not valid for inclined stopes. Silo theory, however, can enhance our understanding of how vertical stresses act within open stopes. Janssen’s method, developed in 1895, and Reimbert’s method, developed in 1953 (and subsequently further developed in 1976), are discussed next.

3.2.6.1 The Janssen Method – Silo Theory

Janssen’s method is based on the equilibrium of a thin horizontal slice, expressed as a differential equation (3-23).

\[
\frac{d\sigma_y}{dy} + \frac{K \cdot \tan(\phi) \cdot \sigma_y}{HR} - \gamma = 0
\]

Figure 3-11: Differential slice in a silo
This yields a solution of the form:

\[
\sigma_y(z) = \left( \frac{\gamma \cdot HR}{K \cdot \tan(\phi)} \right) \cdot \left[ 1 - \exp\left( -\frac{K \cdot \tan(\phi) \cdot z}{HR} \right) \right]
\]

or,

\[
\sigma_y(z) = \sigma_{y_{\text{Max}}} \cdot \left[ 1 - \exp\left( -\frac{K \cdot \tan(\phi) \cdot z}{HR} \right) \right]
\]

Where:
- \( \gamma \) = unit weight of the uncremented rockfill material;
- \( HR \) = hydraulic radius of the stope;
- \( K \) = coefficient of lateral earth pressure;
- \( b \) = Mid-span of the stope (shown in figure 3-11):
- \( \phi \) = rockfill friction angle; and
- \( z \) = rockfill height.

The overall implication of this equation is that below a certain depth in the granular material, all weight supported by the stope walls, via friction, generates either an active or a passive state of stress (as illustrated above).

### 3.2.6.2 The Reimbert Method – Silo Theory

Reimbert’s method is a modification of the Janssen method based on empirical observations. The Reimbert method assumes that the vertical stress, which is the difference between the hydrostatic vertical stress and the load balanced by the frictional force (asymptotic with depth), can be represented by an experimental curve. For a silo undefined between two parallel vertical walls the vertical stress at depth \( z \) is expressed as:

\[
\sigma_y(z) = \gamma \cdot \left\{ z \left[ \left( \frac{z}{Ai} \right) + 1 \right]^{-1} \right\}^{-1}
\]

Where:
Chapter 3 – Review of Design Methodologies

\[ Ai = \frac{2 \cdot L}{\pi \cdot \tan(\phi) \cdot K} \]  
3-27

Combining equations 3-26 and 3-27, yields:

\[ \sigma_y(z) = \gamma \cdot \left\{ \frac{z}{\left( \frac{2 \cdot L}{\pi \cdot \tan(\phi) \cdot K \cdot z} + 1 \right)^{-1}} \right\} \]  
3-28

3.3 Stability of Cemented Rockfill Sill Mats – Analytical Solution

One of the key components of sill mat stability, analyzed previously, is the vertical load exerted by the backfill material. The vertical load is a crucial input parameter of the sill mat stability analytical solution. The specific modes relevant here are: block caving, flexural failure, block sliding (shear failure parallel to the hanging wall within the fill), and rotation failure (tensile/shear failure of the roof as it rotates away from the flat dipping hanging wall). Schematics of these failure modes can be found in Chapter 5.

3.3.1 Caving Failure

Assuming that caving would extend to a stable arch height \( L/2 \) (for a semi-circular arc), all unreinforced sills would form a depth, \( d > L/2 \), and caving would occur when (Mitchell, 1991):

\[ L \cdot \gamma > 8 \cdot \sigma_t / \pi \]  
3-29

Where:

\[ \sigma_t \] = tensile strength of the cemented sill;

\[ \gamma \] = unit weight of the material; and

\[ L \] = stope span.
The driving forces represented by the weight $w$ of the arc of radius $L/2$ are defined as:

$$w = \frac{\pi \cdot L^2}{8} \cdot \gamma$$  \hspace{1cm} 3-30

Sill mat tensile strength $\sigma_t$, over the entire hanging wall to footwall span $L$, represents the resisting force. The factor of safety ($F.S.$) is derived by the forces resisting movement over the driving forces:

$$F.S. = \frac{8 \cdot \sigma_t}{\gamma \cdot \pi \cdot L}$$ \hspace{1cm} 3-31

### 3.3.2 Flexural Failure

A wide, thin sill mat is susceptible to flexural failure due to the relatively low tensile strength of cemented backfill. Using standard flexural formulae for a fixed-end uniformly-loaded beam, failure is predicted when (Mitchell, 1991):

$$\left( \frac{L}{d} \right)^2 > \frac{2 \cdot (\sigma_t + \sigma_c)}{w}$$ \hspace{1cm} 3-32

or:

$$\left( \frac{L}{d} \right)^2 > \frac{2 \cdot (\sigma_t + \sigma_c)}{\sigma_v + d \cdot \gamma}$$ \hspace{1cm} 3-33

Where:
- $\sigma_t$ = tensile strength of the cemented sill;
- $\sigma_c$ = horizontal confining stress;
- $w$ = uniform loading, which includes the self-weight of the sill mat;
- $L$ = stope span; and
- $d$ = sill depth.
3.3.3 **Sliding Failure**

From equilibrium, block sliding of the sill as a result of side shear failure occurs when (Mitchell, 1991):

\[
(\sigma_v + d\cdot\gamma) > 2 \cdot \left( \frac{\tau_i}{\sin^2(\beta)} \right) \cdot \left( \frac{d}{L} \right)
\]

Where:
- \(\tau_i\) = shear strength of the cemented sill; and
- \(\beta\) = dip of the stope.

3.3.4 **Rotational Failure**

Rotational failure is most likely to occur when the shearing resistance at the hanging wall contact is too low, as a result of poor quality hanging wall rock and/or low dip angles. Separation subsequently occurs. For low dip angles, an approximate prediction can be made by assuming the sill mat tensile failure results in a complete separation in hanging wall contact (i.e., a gap forms).

Rotational failure develops when (Mitchell, 1991):

\[
(\sigma_v + d\cdot\gamma) > \frac{d^2 \cdot \sigma_t}{2 \cdot L \cdot (L - d \cdot \cot(\beta)) \cdot \sin^2(\beta)}
\]

Technically, this formula has an embedded factor of safety of 2.0. The formula should therefore be of the form:

\[
(\sigma_v + d\cdot\gamma) > \frac{d^2 \cdot \sigma_t}{L \cdot (L - d \cdot \cot(\beta)) \cdot \sin^2(\beta)}
\]

3.3.5 **Strength Properties**

The tensile strength of the fill is crucial in its capacity to resist damage due to blast vibration, and to resist failure where undercutting is effected. Tensile strength of the fill is
limited to, and is primarily a function of, binder content. Resistance to blast vibration damage is a function of dynamic tensile strength, which can be approximated as 5% to 15% of unconfined compressive strength. Arioglu (1983) concluded that tensile strength of cemented aggregate fill occurs at approximately 15% of UCS. Smith (1982) concluded that tensile strength of cemented tailings fill occurs at approximately 12% of UCS. Yu (unpublished) states that the actual strength of CRF placed in a mine will be approximately 2/3 of the laboratory value that is obtained from standard 6 inch diameter concrete test cylinders, but will be about 90% of the value obtained from 12-inch diameter cylinders.

For stability analysis of CRF sill mats, this study uses a tensile strength of 10% of UCS. All other strength properties were obtained using Mohr-Coulomb stress strength criteria.

The strength properties used to calculate the factors of safety are obtained by the analytical equations:

\[
\text{Cohesion, } c = \frac{UCS}{2} \cdot \tan(45 + \frac{\phi}{2}) \tag{3-37}
\]

\[
\text{Tensile Strength, } \sigma_t = \frac{UCS}{10} \tag{3-38}
\]

\[
\text{Shear Strength, } \tau_s = c + \sigma_n \cdot \tan(\phi) \tag{3-39}
\]

\[
\text{Normal Stress, } \sigma_n = \frac{1}{2} \cdot K_a \cdot \gamma \cdot d^2 \cdot \sin(\beta) \tag{3-40}
\]

### 3.4 Empirical Stability Design Methods

This thesis also addresses the stability of open stopes adjacent to backfill. Empirical stability design methods are the basis for the development of a solution for stopes with backfilled abutments.
3.4.1 The Stability Graph Method

The Stability Graph Method for open stope design was initially proposed by Mathews, et al. (1981), depicted in figure 3-12, and subsequently modified by Potvin (1988), using 242 case histories (176 unsupported, 66 supported) and by redefining some of the adjustment factors. The result was the modified stability number $N'$. The influence of cable bolt support was re-examined by Potvin & Milne (1992), depicted in figure 3-13, and Nickson (1992), figure 3-14, to arrive at the Modified Stability Graph. Stability was qualitatively assessed as being: stable, potentially unstable, or caved.

Research at the University of British Columbia (1988) quantified the degree of slough by the introduction of the term “Equivalent Linear Overbreak/Slough,” ELOS, (Clark, 1998) as depicted in figure 3-15. Empirical estimation of wall slough employs:

- Stability Number $N$ (or $N'$, i.e., modified); and

![Figure 3-12: Mathews Stability Graph (Mathews et al., 1981)](image-url)
Chapter 3 – Review of Design Methodologies

Figure 3-13: Modified Stability Graph (Potvin and Milne, 1992)

Figure 3-14: Modified Stability Graph with support (Nickson, 1992)
3.4.2 The Stability Graph Method – Radius Factor

The Stability Graph, later augmented by Mah (1997), was first introduced in terms of Radius factor, by Milne (1997). Radius factor accounts for complex mining geometries whereas Hydraulic Radius is restricted to rectangular surfaces (Area/Perimeter). The radius factor has the same form as hydraulic radius; however, instead of being based on the average of four measurements of the supporting abutments, several measurements are taken at small angular increments, as shown in figure 3-16(b). This allows greater flexibility in assessing the geometry of the support potential at a specific point on the surface (Milne, 1997), involving complex shapes, providing “weightings” (i.e., RF values).

The Stability Number N' employed in figure 3-16(a) is similar to that of figure 3-15. The only significant difference is in the calculation of stope surface geometry, which employs the radius factor parameter.

Figure 3-16: Empirical estimation (a) of wall slough (ELOS) expressed in terms of Radius Factor (b)

3.5 Software and Device Utilized

3.5.1 Flac\textsuperscript{2D} – Two Dimensional Explicit Finite Difference Method\textsuperscript{1}

Flac\textsuperscript{2D} (version 4.0) is the numerical modeling tool used in this study to examine the load exerted by backfill. Flac\textsuperscript{2D} (Fast Lagrangian Analysis of Continua) is a two-dimensional explicit finite difference program used in engineering mechanics computations.

The program can simulate the behavior of structures built of soil, rock, or other materials, which may undergo plastic flow when their yield limits are reached. Materials are represented by elements, or zones, which form a grid that can be adjusted to fit the shape of the object to be modeled.

Each element behaves according to a prescribed linear or nonlinear stress/strain law in response to the applied forces or boundary restraints.

The material can yield and flow and, if set to do so, the grid can deform and move the material that is represented. Plastic collapse and flow are modeled very accurately using the explicit Lagrangian calculation scheme and the mixed-discretion zoning techniques in Flac\textsuperscript{2D}. Flac\textsuperscript{2D} is also the numerical modeling tool used to derive the design charts for stability of the CRF sill mat. Flac\textsuperscript{2D} is an effective tool for modeling non-linear gravity
driven materials like uncedmented and cemented rockfill. Within appropriate quality control
guidelines, the material (of required strength for a self-weight supported structure like the
CRF sill mat) should behave as an inelastic material, once cured. Even so, the two
dimensional version of Flac$^{2D}$ will overestimate results given that the third dimension is
considered to be infinite, and therefore Flac$^{2D}$ is considered to provide a conservative
estimate.

At Musselwhite, the stope strike length, where sill mats were constructed, is typically
three or more times greater than the stope height, and the use of plain-strain numerical
modeling is effective and practical, for understanding its behavior. An essential aspect of
modeling backfill using Flac$^{2D}$ lies in the fact that backfill is deposited as a material with no
initial stress state. The only stress comes from its own weight, which allows the distribution
of vertical loads to be a function of Hangingwall (HW) and Footwall (FW) host-rock/backfill
interactions, and backfill properties.

Another important feature of Flac$^{2D}$, for the purpose of this analysis, is its capacity to
display actual deformations of grid zones caused by material failure under different stress
states and material properties. The explicit, time-marching solution permits analysis of
progressive failure and collapse, significant phenomena in studies of mine design.

It is also possible to perform parametric studies, conveniently, by restarting the analysis
at a previous state. Results are given in Chapter 5.

†1 technical source: Itasca Consulting Group Ltd. (2002)

3.5.2 Map3D - Three Dimensional Boundary Element Method$^{2}$

Map3D is a comprehensive, fully three-dimensional rock stability analysis package. Map3D
can construct models as well as analyze and display displacements, strains, stresses,
and strength factors. Map3D is effective for modeling rock engineering design problems
involving both large tabular orebodies and irregular massive excavations.

The program features:

- full three-dimensional stress analysis;
- tabular stress analysis with yielding pillars;
- elastic, non-linear, creep and thermal/fluid flow options;
- fault slip and fracture analysis capability;
Chapter 3 - Review of Design Methodologies

- simulation of stiff dykes, weak schist zones and backfill;
- external loading effects;
- structural support placement; and
- seismic database visualization.

Models may be made of rock slopes, open pits, tunnels, fractures, and underground excavations, with yielding (i.e., non-linear) zones of different moduli (e.g., stiff dykes or soft ore zones) and loads resulting from steady-state thermal/fluid flow. Excavations can be intersected by multiple discrete faults (i.e., non-planar and gouge filled) that slip and open. Map3D can simulate ground support elements such as arches, steel sets, props, thick liners, chalks, strong backfill, etc.

Map3D is based on three-dimensional boundary element formulation. Both displacement discontinuity and force discontinuity (fictitious force) formulations are available in the software package. Special proprietary elements are used for simulation of thermal/fluid flow and non-linear effects. Because automated lumping is built into the program, Map3D can accommodate more than 300,000 elements (and over 1,000,000 degrees of freedom) using merely desktop computers. Larger models can be created as required.

†2 technical source: Mine Modeling Pty Ltd. (2004)

Map3D-SV (standard version), release 5.0, is used in this study to calculate the displacements on the stope wall for different strike lengths. The results and analysis are given in Chapter 6.

3.5.3 NeuroShell Predictor – Neural Networks†3

Neural network technology mimics the brain's own problem solving system. Just as humans apply knowledge gained from past experience to new problems or situations, a neural network takes previously solved examples to build a system of "neurons" that makes new decisions, classifications, and forecasts.

A Neural network looks for patterns within training sets of data, then learns these patterns, and develops the ability to correctly classify new patterns, or to make forecasts and predictions. Neural networks excel at problem diagnosis, decision making, prediction,
classification, and other problems where pattern recognition is vital but precise computational answers are not required.

The neural network begins by evaluating linear relationships between the inputs and the output. Weights are assigned to the links between the input and output neurons. After those relationships are identified, neurons are added to the “hidden layer”, so that non-linear relationships can then be searched for. Input values in the first layer are multiplied by the weights and passed to the second (hidden) layer. Neurons in the hidden layer “fire,” i.e., produce outputs that are based on the sum of the weighted values passed to them. The hidden layer passes values to the output layer in the same fashion, and the output layer produces the desired results, i.e., its predictions.

The network “learns” by re-adjusting the weights between layers. The answers the network produces are continually compared with the correct answers, and each time the connecting weights are adjusted slightly in the direction of the correct answers. Additional hidden layer neurons are added as needed to isolate structures in the data set.

Eventually, if the problem can be learned, a stable set of weights evolves and will produce useable answers for all sample decisions, or predictions. The real power of neural networks becomes evident when the trained network is able to produce useable results for data that the network has never “seen” before.

†3 technical source: Ward System Group Inc. (1997)

NeuroShell Predictor version 5.1 is used in this study to compare ELOS (Equivalent Linear Overbreak Slough) data for transverse primaries, or isolated stopes, against longitudinal stopes, where backfill is adjacent to the exposed wall. Results are given in Chapter 6.

3.5.4 Unwedge – Underground Wedge Stability Analysis

Unwedge (version 2.2) is a three-dimensional stability analysis and visualization program for underground excavations in rock, containing intersecting structural discontinuities. Safety factors are calculated for potentially unstable wedges so that support requirements can be modeled using various types of pattern and spot bolting, and shotcrete. Unwedge can perform safety factor analysis, in accordance with reinforcements employed, and interpret the results.
The graphical data interpreter provides a rich set of tools, including 3D animation, providing for convenient display of wedges surrounding the excavation.

*Unwedge* incorporates enhanced support models for bolts, shotcrete, and support pressures, and has the ability to optimize tunnel orientation. There is also an option to examine different combinations of three joint sets based on a list of multiple joint sets. *Unwedge* uses a new analysis engine based on Goodman and Shi's block theory, including the ability to evaluate induced stress around the excavation and its effects on stability. *Unwedge* includes new strength models such as Barton-Bandis and Power Curve, and the ability to improve the scaling and sizing of wedges.

†4 technical source: Rocscience Inc.(2004)

*Unwedge* is used in this study to represent structural blocks, and support pattern performance in a typical production tunneling scenario.

### 3.5.5 Dips – Graphical and Statistical Analysis of Orientation Data

*Dips* is designed for the interactive analysis of orientation-based geological data and allows the user to analyze and visualize structural data, following the same techniques used in manual stereonets. Additionally, it has several computational features, such as statistical contouring of orientation clustering, mean orientation and confidence calculation, cluster variability, and qualitative and quantitative feature attribute analysis.

*Dips* is designed for analysis of parameters related to the engineering analysis of rock structures, however, the free format of the Dips data file permits analysis of any orientation-based data (Rocscience Inc., 2004).

†4 technical source: Rocscience Inc.(2004)

Dips version 5.0 is used in this study in the modeling of Musselwhite's fabric analysis (see 4.2).

### 3.5.6 Methods Ground Control Assessment - Cavity Monitoring System Surveys

Optech's Cavity Monitoring System (CMS) is an advanced, reflectorless, laser-based auto scanning system that provides efficient and accurate measurement of dangerous and inaccessible cavities. CMS collects thousands of data points per minute. The data is used to
determine stope volume and stope dilution, sloughing/backfill volumes and mine measurements, and to create detailed profiles of pillars, orepasses, raises, drifts, and drawpoints. Mine surveyors obtain detailed 3D survey data images of underground production stopes. Mine planners use the data to determine the exact area mined, and quantify volumes of mined/unmined ore. Geologists use the data to match month end reconciliation. Rock mechanics then use the data to predict rock failure (e.g., hangingwall and footwall).

After a set of blast rings is designed, blasted, excavated and surveyed, the results are compared to the original blast design. The results are then analyzed for tonnage of lost ore (underbreak), tonnage of waste rock or unplanned dilution (overbreak), and accuracy of grades sent to the mill (reconciliation). Ultimately, the results are analyzed for the success, or lack of success, of drilling and blasting, and to predict dilution in future stopes (Optech Inc.).

†5 technical source: Optech Incorporated.(2004)
Chapter 4 – Database

4 DATABASE

4.1 Intact Strength

A total of 32 additional uniaxial compressive strength tests were carried out, augmenting the existing 12 results on both iron formation and intermediate-mafic/felsic volcanic rocks. Four more samples of the iron formation were tested with extensometers, allowing for elastic modulus determination. The tests were performed as recommended in the ISRM Suggested Methods (Brown, E.T. (Editor), 1981). The test result averages are presented in Table 4-1 and Table 4-2.

Table 4-1: Mean uniaxial compressive strength, standard deviation and ISRM hardness

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Number of Samples</th>
<th>UCS (MPa)</th>
<th>Standard Deviation (MPa)</th>
<th>ISRM Hardness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Iron Formation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4ea</td>
<td>7</td>
<td>194</td>
<td>46</td>
<td>Very Strong Rock – R5</td>
</tr>
<tr>
<td>4f</td>
<td>12</td>
<td>92</td>
<td>31</td>
<td>Strong Rock – R4</td>
</tr>
<tr>
<td>4e</td>
<td>3</td>
<td>116</td>
<td>15</td>
<td>Very Strong Rock – R5</td>
</tr>
<tr>
<td>4b</td>
<td>5</td>
<td>156</td>
<td>46</td>
<td>Very Strong Rock – R5</td>
</tr>
<tr>
<td>4e4f</td>
<td>3</td>
<td>86</td>
<td>11</td>
<td>Strong Rock – R4</td>
</tr>
<tr>
<td>Intermediate-Mafic Volcanic</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AVol</td>
<td>9</td>
<td>181</td>
<td>43</td>
<td>Very Strong Rock – R5</td>
</tr>
<tr>
<td>Intermediate-Felsic Volcanic</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BVol</td>
<td>5</td>
<td>117</td>
<td>22</td>
<td>Very Strong Rock – R5</td>
</tr>
</tbody>
</table>
Table 4-2: Mean Young's Modulus (E) and Mean Poisson's Ratio (v)

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Young Modulus E (GPa)</th>
<th>Standard Deviation (GPa)</th>
<th>Poisson's ratio v</th>
<th>Standard Deviation (v)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4ea</td>
<td>84</td>
<td>13</td>
<td>0.21</td>
<td>0.03</td>
</tr>
<tr>
<td>4f</td>
<td>72</td>
<td>7</td>
<td>0.21</td>
<td>0.02</td>
</tr>
<tr>
<td>AVol</td>
<td>85</td>
<td>N/A</td>
<td>0.23</td>
<td>N/A</td>
</tr>
<tr>
<td>BVol</td>
<td>79</td>
<td>N/A</td>
<td>0.24</td>
<td>N/A</td>
</tr>
</tbody>
</table>

The tests were performed using an electro-hydraulic servo-controlled stiff testing machine, used for comparing the various physico-mechanical properties of rocks under different conditions of loading to ISRM standards.

### 4.2 Fabric Analysis

Generally, three joint sets are prominent in all rock types: parallel to the ore, perpendicular to the ore, and flat. Some random, typically vertical, faulting also exists as does the 4f (garnet-biotite-schist), which is also vertical and parallel to the ore zone. A structurally intense domain seems to be characteristic of the BVol (footwall drift) in which the three prominent joint sets exist, except with tighter spacing, resulting in blocky ground. The AVol has characteristically tight foliation (north-south, vertically dipping), which ranges in spacing from below one centimeter to five centimeters, or more. Flat and crosscutting joints in the AVol are spaced from one to two meters or more as opposed to the BVol. Ground conditions at the back of the AVol are generally very good. Because there is tight foliation, the walls are prevented from buckling.

The ore zone consists of three predominant joint sets in dip/dip direction, global orientation format:

- $83^\circ / 90^\circ$ (north-south, vertical joints);
- $75^\circ / 177^\circ$ (east-west, vertical joints); and
- $25^\circ / 210^\circ$ (flat jointing).
Figure 4-1 portrays the equal area projection stereonet, obtained by employing Rocscience’s *Dips* program on the three major joint sets.

![Equal Area Stereonet](image)

**Figure 4-1: Equal area stereonet representing the three major joint sets**

In the ore zone, the three joint sets typically have an average spacing of 1 - 1.5 meters. The joint sets have slightly rough surfaces and little or no separation with hard joint wall rock. In the BVol, the joints are typically spaced more tightly and have thin chloritic-coated surfaces.

There is the potential for wedge or block failures in these areas, both when drifting and when longholing. Wedge failures may occur at the hangingwall or the back of the open stope. Wedges in the drifts are normally supportable with standard 1.8 meter (#6) support.

Figure 4-2 portrays a 10-tonne wedge, supported by standard 1.4 meter by 1.4 meter pattern, fully-grouted resin rebar. The factor of safety for the standard bolting pattern employed at Musselwhite produces a high degree of confidence. Stability analysis was effected using Rocscience’s *Unwedge* program on three major joint sets (previously defined) with 1 meter spacing each, considering no cohesion and a 35 degree friction angle for the joint sets’ infilling material. The tunnel trends to the north.
Failures in the open stope due to random faulting and jointing produce the potential for unplanned dilution. Some failures of up to 4.5 meters deep have been observed occasionally, due to the poor hangingwall rock mass quality 4f, which runs parallel to the hangingwall.

4.3 **Rock Mass Analysis**

Bieniawski's Geomechanics Classification (1976), or the Rock Mass Rating (RMR), of the stopes varies between 40% - 70% for the hanging wall, 60% - 80% for the footwall, 65% - 70% for the back of the stope, and 70% - 75% for the ore itself. Table 4-3 illustrates an example of the RMR where R4 rock strength represents uniaxial compressive strength between 100 and 250 MPa, and the Rock Quality Designation (RQD) provides a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100 mm (4") within the total length of the core. Barton's Tunneling Quality Index Q, was also measured, or derived, for each stope, to estimate its stability and determine the approximate degree of hangingwall and footwall slough.
Table 4-3: RMR and Q' of a typical footwall stope

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Rating (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength</td>
<td>R4</td>
<td>12</td>
</tr>
<tr>
<td>% RQD</td>
<td>60-70%</td>
<td>12-14</td>
</tr>
<tr>
<td>Joint Spacing</td>
<td>0.25-0.5m</td>
<td>11-17</td>
</tr>
<tr>
<td>Joint Condition</td>
<td>Slightly rough/smooth</td>
<td>15-17</td>
</tr>
<tr>
<td>Ground Water</td>
<td>Dry</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>Total RMR</td>
<td>60-70%</td>
</tr>
<tr>
<td></td>
<td>Total Q'</td>
<td>5.9-18.0</td>
</tr>
</tbody>
</table>

The following table summarizes the average range of Rock Mass Rating (Bieniawski, 1976) for the different rock types found at Musselwhite throughout the life of the mine.

Table 4-4: Typical RMR ranges for the different Musselwhite rock types

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>BVol</th>
<th>AVol</th>
<th>4f</th>
<th>4b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength</td>
<td>120</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>(240 MPa)</td>
<td>(180 MPa)</td>
<td>(100 MPa)</td>
<td>(150 MPa)</td>
</tr>
<tr>
<td>RQD</td>
<td>18</td>
<td>18</td>
<td>17-10</td>
<td>17-10</td>
</tr>
<tr>
<td></td>
<td>(75-90)</td>
<td>(75-90)</td>
<td>(50-90)</td>
<td>(50-90)</td>
</tr>
<tr>
<td>Spacing</td>
<td>18-13</td>
<td>18-13</td>
<td>11-8</td>
<td>11-8</td>
</tr>
<tr>
<td></td>
<td>(0.3-1m)</td>
<td>(0.3-1m)</td>
<td>(50-300mm)</td>
<td>(50-300mm)</td>
</tr>
<tr>
<td>Condition</td>
<td>17-12</td>
<td>17-12</td>
<td>15-7</td>
<td>15-7</td>
</tr>
<tr>
<td></td>
<td>(tight-sep&lt;1mm)</td>
<td>(tight-sep&lt;1mm)</td>
<td>(tight-slick)</td>
<td>(tight-slick)</td>
</tr>
<tr>
<td>Ground Water</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>(dry)</td>
<td>(dry)</td>
<td>(dry)</td>
<td>(dry)</td>
</tr>
<tr>
<td>RMR</td>
<td>75-65</td>
<td>75-65</td>
<td>65-47</td>
<td>65-47</td>
</tr>
<tr>
<td></td>
<td>(typical:70)</td>
<td>(typical:70)</td>
<td>(typical:60)</td>
<td>(typical:55)</td>
</tr>
</tbody>
</table>
Some of the RMR points were derived by interpreting geological sections using the Vulcan geological database. Thus, the contact rock type and thickness was associated with the RMR range in each case (see Table 4-4).

![Geological section obtained using Vulcan database](image)

**Figure 4-3: Geological section obtained using Vulcan database**

### 4.4 Stress

The following table summarizes the stress state of Musselwhite orebodies at a depth of 500 meters (Arjang et. al., 1997):

<table>
<thead>
<tr>
<th>Type</th>
<th>In-situ Stresses</th>
<th>e.g., at 500m</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{h_{\max}}(E-W)$</td>
<td>$2.5 \times \sigma_v$ MPa/m depth</td>
<td>38MPa</td>
</tr>
<tr>
<td>$\sigma_{h_{\min}}(N-S)$</td>
<td>$1.5 \times \sigma_V$ MPa/m depth</td>
<td>23MPa</td>
</tr>
<tr>
<td>$\sigma_v$(Vertical)</td>
<td>$0.03$ MPa/m depth</td>
<td>15MPa</td>
</tr>
</tbody>
</table>

### 4.5 Numerical Modeling – Constitutive Models and Material Properties

Two different constitutive models were used to characterize both material types, host rock and backfill. Host rock material was assigned an elastic and isotropic model. This model provides the simplest representation of material behavior. The model is valid for
homogeneous, isotropic, continuous materials that exhibit linear stress-strain behavior. Backfill material was represented by the Mohr-Coulomb model, which is the conventional model used to represent shear failure in soils and rocks.

Table 4-6: Mohr-Coulomb stress-strain and strength parameters for backfill and host rock

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Rockfill</th>
<th>Host Rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight (kN/m3), $\gamma$</td>
<td>19.67</td>
<td>26.48</td>
</tr>
<tr>
<td>Cohesion (MPa), $c$</td>
<td>0.0</td>
<td>-</td>
</tr>
<tr>
<td>Friction angle (degree), $\phi$</td>
<td>37.0</td>
<td>-</td>
</tr>
<tr>
<td>Tensile strength (MPa), $\sigma_t$</td>
<td>0.0</td>
<td>-</td>
</tr>
<tr>
<td>Young modulus (MPa), $E$</td>
<td>300.0</td>
<td>74,000</td>
</tr>
<tr>
<td>Poisson’s ratio ( ), $\nu$</td>
<td>0.25</td>
<td>0.22</td>
</tr>
<tr>
<td>Bulk modulus (MPa), $K$</td>
<td>200</td>
<td>44,047</td>
</tr>
<tr>
<td>Shear modulus (MPa), $S$</td>
<td>120</td>
<td>30,327</td>
</tr>
</tbody>
</table>

4.6 Empirical Stope Design – Musselwhite’s Stability Database

The stability of underground excavations at Musselwhite is determined by three components:

- characterization of the surrounding rock mass;
- geometry and relative orientation of the opening; and
- induced stress-state about the opening.

The first component is the identification and quantification of the rock mass properties, e.g., rock type, support, RMR, and/or Barton’s Tunneling Quality Index Q. The second component is the size of the planned opening and the dip of the stope, i.e., strike length and slanted height of the opening. From this, the hydraulic radius (or radius factor) of the exposed wall is determined. The third aspect considered is the degree of relaxation of the stope wall and the major principal stress (versus minor principal stress) about the opening. The modified stability number $N'$ is adjusted for the new stress conditions.

By identifying and quantifying rock mass properties and the planned geometry it is possible to derive empirically the stability of the stope, using the Modified Stability Graph.
(Potvin, 1998) and/or determining the amount of wall slough using the Equivalent Linear Overbreak Slough (ELOS) Graph (Clark, 1998).

Table 4-7 portrays Musselwhite’s mine stability database, comprised of the 69 stopes investigated. For each stope, the following data are indicated: length, height, dip, RMR, and ELOS, for both hangingwall and footwall. The ELOS value was calculated by comparing the drill layout and/or geological block model with the CMS survey profile for each stope. The following table was derived specifically for this study:
### 4.7 Statistical Analysis of the Musselwhite Database

The following figures portray frequency of occurrence histograms from the Musselwhite mine database. These histograms include a normal distribution curve for each parameter. The histograms were sorted into hangingwall and footwall data, except for stope length, which was considered as being unique for each stope.
Figure 4-4: Statistical Analysis on Musselwhite stability database
From the previous statistical analysis, it is obtained the ranges of stope configurations (height, span and dip) which will be used in the following chapters as input parameters for numerical modeling and neural network analysis.
5 DESIGN GUIDELINES FOR CEMENTED ROCKFILL SILL MATS

5.1 Introduction

Theories of cemented rockfill sill mat design demand the optimization of fill requirements, achieving the strength needed, while meeting safety guidelines and minimizing mining costs.

Cement is the largest material cost component of backfill. Therefore, strategies aimed at reducing cement content are of the highest benefit to mining operations. This chapter defines and compares the relevant analytical relationships and numerical modeling results of backfill behavior on sill mat stability. The analysis is done via observations of practitioners and researchers working in the mining field.

Note that more than 1000 numerical models have been executed, as a part of this thesis, that directly relate to these analytical relationships, including models to determine vertical stress and models to determine the stability of cemented rockfill mats.

5.2 Load of Backfill

One of the key factors in assessing cemented rockfill stability is quantifying the vertical stress acting on the top of the sill mat. In situ measurements and analytical predictions derived by various researchers imply that arching of unconsolidated rockfill material decreases the vertical load on the sill by transferring part of the total load to the hangingwall and footwall. Methods to better estimate the true load lead to more reliable and cost effective solutions in the design of sill mats. Overestimation of the vertical load produces unnecessary expense by increasing the quantity of cement used and/or the height of the sill mat required to withstand the supposed load. Underestimating can cause a premature failure of the sill mat while mining is being conducted below it, resulting in ore lost, ore dilution, cycle interruption, damage to equipment, and a compromise in personal safety.

Theoretical considerations derived from Janssen (1895), Terzaghi (1948), Reimbert (1976), and Blight (1984) are compared with numerical modeling results to determine the theoretical true load exerted by the fill. To analyze backfill load, first it is necessary to determine accurately the coefficient of lateral earth pressure, $K$, an input variable of the analytical equation. Failing to estimate accurately the value of $K$ produces unreliable vertical
load calculations. The derived curve for $K$ can be used when numerical modeling software is not available and/or to compare vertical loads corresponding to different $K$ values.

### 5.2.1 Coefficient of Lateral Earth Pressure $K$

This thesis determines the coefficient of lateral earth pressure $K$ using numerical modeling and proposes that an effective computational method is capable of representing the behavior of cohesionless soils. Moreover, the software applied here can be used to characterize the deformations of backfill using only the effect of gravity and its interaction with the stope walls. The computational tool employed is the finite difference program known as **Flac**.  

Figure 5-1 illustrates a Flac graph of stress measurements at a specific point within a backfilled stope. The stope is backfilled in layers of 5 meters each, to a maximum height of 35 meters. The $y$-axis corresponds to the horizontal stress, $\sigma_{xx}$, and the $x$-axis corresponds to the vertical stress, $\sigma_{yy}$. The resultant slope represents the coefficient of lateral earth pressure, $K$.

Figure 5-1: Coefficient of lateral earth pressure $K$ for cohesionless material

Figure 5-2 displays the graph of the coefficient of lateral earth pressure at different friction angles, comparing the results of analytical equations of the coefficients for the active
(Ka), and at-rest (Ko) cases, taking into consideration the settling effect (Ks), with the results obtained from numerical modeling.

![Graph](image)

**Figure 5-2: Comparison of analytically and numerically determined coefficients of lateral earth pressure**

Figure 5-2 portrays the values obtained using numerical modeling. The values are consistent with the active case, up to the point where the friction angle reaches 30 degrees. At friction angles greater than 30 degrees, the $K$ value becomes constant, and the curve tends toward the at-rest condition. This behavior might occur because at lower friction angles greater deformation of the material takes place. The rockfill material behaves as if the wall is in motion, re-forming itself by the action of gravity, and the material is supported by the stope walls, generating the active state of stress. On the other hand, at friction angles greater than 30 degrees the deformation tends to decrease and the interaction between rockfill and the stope walls decreases accordingly, tending toward the at-rest state of stress.

Figure 5-3 and figure 5-4 portray the second order polynomial curve that best fits the data obtained using numerical modeling ($K$ Derived). The coefficient of lateral earth pressure is plotted against the sine of the friction angle and the best fit curve determined.
Figure 5-3: Coefficient of lateral earth pressure obtained using numerical modeling and best fit curve derived

Figure 5-4: Coefficient of lateral earth pressure obtained using numerical modeling and best fit curve derived for the rockfill friction angle range

Equation 5-1 represents the best fit curve:

\[ K = 1.4 \cdot \sin^2(\phi) - 2 \cdot \sin(\phi) + 1 \]  

5-1

Note that equation 5-1 is only applicable for cohesionless backfill material with friction angles ranging from 0 degrees to 40 degrees.
Figure 5-5 portrays the variation of the slope \((K)\) of the horizontal versus vertical stress for different rockfill friction angles.

![Horizontal Stress vs. Vertical Stress at different Friction Angles](image)

**Figure 5-5: Horizontal vs. vertical stress at different rockfill friction angles**

### 5.2.1 Vertical Stress Comparison using different Coefficients of Lateral Earth Pressure

Variations in vertical stress obtained via different coefficients of lateral earth pressure \(K\) are portrayed by figure 5-6. It shows the plot of maximum vertical stress for the active case, the at-rest case, and the derived case using numerical modeling while considering the following backfill properties and stope configuration:

- \(\gamma = 19.67 \text{ kN/m}^3\)
- \(\phi = 37.0^\circ\)
- \(L = 10.0 \text{ m}\)
- \(\beta = 90^\circ\)
As portrayed in figure 5-6, there is a considerable difference between the maximum vertical pressure using the active versus the at-rest coefficients of lateral earth pressure. The variation of approximately 200 kPa can be attributed to the difference between the designs of a stable versus an unstable sill mat. Results obtained using numerical modeling produce an intermediate value, which falls between the active and the at-rest cases. The most important outcome here is that the solution obtained using numerical modeling accounts for the deformation of the material within the stope. The behavior of this cohesionless material is affected by the magnitude of the friction angle at the contact between the stope walls and the rockfill material.

5.3 Vertical Stress Using Numerical Modeling

5.3.1 Introduction

This section addresses the total load on the floor of the stope given that it is this load that determines the stability of CRF sill mats.

Rockfill material deposited into stopes constitutes a granular, or pseudo-solid, mass, for which equilibrium laws still apply. Given a plastic, homogeneous, and isotropic material, a solution can be derived from numerical modeling results, and is here so derived for this case.
The problem defined by the equilibrium of confined rockfill material is complex, and does not well fit the theory of elasticity. It is extremely difficult to determine accurately the loads applied to the stope walls and floor via existing theories. A numerical modeling solution capable of accounting for the friction that exists between the fill and the stope walls, and so making possible comparison of the resultant curves with existing analytical equations, is enormously valuable.

The process of modeling backfill as a function of the fill height proceeds as follows: a five meter Hangingwall (HW) to Footwall (FW) layer of backfill is deposited on the floor of the stope, the model is executed, and the vertical and horizontal loads are then measured. Incrementally, five meter layers of backfill are added on top of previous layers. The model is executed and loads are measured for each increment to the point where the stope has been filled with seven layers of backfill material.

This analysis does not incorporate wall convergence, under the assumption that by the time waste rock is backfilled, wall displacement has already occurred.
Figure 5-7 (a) and figure 5-7 (b) portray two models backfilled at two different stages: 25 meters and 35 meters of backfill. The vertical stress contour represents the stress exerted by the cohesionless backfill material with a 37 degree friction angle. From this, the arching of the fill due to the material’s frictional component and its interaction with the stope walls can be seen.

Figure 5-7: 25 meters (5 layers) and 35 meters (7 layers) of unconsolidated backfill deposited in the stope.
5.3.2 Stress Distribution as a Function of Stope Dip Angle

Various stope geometries are analyzed to determine the maximum stress and the stress distribution along the stope span. Variation in stope dip angle is modeled to analyze the effects on stress and on stress distribution. The stress distribution along the stope span for a range of stope dip angles is portrayed by figure 5-9. Note that the vertical stress decreases as the stope dip decreases, and the maximum stress tends to locate near the stope’s footwall. The importance of this graph is in that it produces a reference point as to where the material’s center of mass should be located.

Figure 5-8: Vertical stress distribution along span at different stope dips

Figure 5-9 portrays the stress contour of a stope dipping at 70 degrees.
Figure 5-9: Stress contour for a stope dipping at 70 degrees

5.3.3 Vertical Stress as a Function of Stope Span

Figure 5-10 portrays the vertical stress of various rockfill heights for different stope spans. This graph, and those to follow, deal with vertical stopes. The graphs dealing with inclined stopes appear later. Note that vertical stress is recorded at mid-span, on the floor of the stope.

Figure 5-10: Vertical stress at various stope spans for different rockfill heights
The numerical results predict a decrease in vertical stress as rockfill height increases, as a result of arching of the material as it interacts with the stope walls. Similar asymptotic behavior can be seen in the analytical solutions derived by Janssen and Reimbert. For two stope spans, 10 and 12 meters, the fill heights extend beyond 35 meters, as these are possible scenarios for Musselwhite’s constructed sill mats.

5.3.4 Vertical Stress as a Function of Rockfill’s Density

Several models are executed evaluating the behavior of unconsolidated rockfill for different material properties. The results are compared with the analytical formulations given in Chapter 3. The following sections discuss vertical stress variation for each parameter assessed. The analytical and numerical results both imply linear behavior at varying rockfill densities, as indicated in figure 5-11.

![Vertical Stress vs. Rockfill Density at different Rockfill Heights](image)

Figure 5-11: Vertical stress vs. rockfill density at varying rockfill heights

5.3.5 Vertical Stress as a Function of Rockfill’s Friction Angle

Figure 5-12 portrays the variation of vertical stress at different friction angles, for a given height. As expected, for friction angles approaching 0 degrees, the vertical stress is equivalent to the hydrostatic pressure of a material of 2t/m³ density. The higher the friction angle of the material, the lower is the load exerted on the floor of the stope.
Figure 5-12: Vertical stress vs. rockfill height at varying friction angles

5.3.6 Comparison of Analytical and Numerical Methods to Determine Vertical Stress

The following figures compare the vertical stress exerted on the floor of a backfilled stope using analytical and numerical results. Figure 5-13 displays the curves of vertical stress at different rockfill heights, comparing Janssen’s and Reimbert’s analytical equations with Flac$^{2D}$ results. Figure 5-14 displays the analytical formulations derived by Mitchell, Blight, Janssen, and Reimbert, and compares them to numerical results at different stope spans. Figure 5-15 compares vertical stresses for a 10 meter stope span, for each method. Figure 5-15 illustrates that there are substantial differences in vertical stress calculations among the different methods analyzed.

Note the 0.12 MPa difference between Flac$^{2D}$ and the most commonly used method, Mitchell (1991). This difference implies over-estimation of the strength required, when using Mitchell’s method. This thesis proposes that numerical modeling is the most reliable method, as it accounts for deformation of cohesionless material, as well as interaction between the material and the stope walls. Reimbert’s solution yields very similar results as those of numerical modeling, but it is limited in that it was developed for use with vertical silos.
Chapter 5 - Design Guidelines for Cemented Rockfill Sill Mats

Figure 5-13: Vertical stress over rockfill height comparing analytical and numerical results

Figure 5-14: Vertical stress versus stope span using different methods
Chapter 5 – Design Guidelines for Cemented Rockfill Sill Mats

Figure 5-15: Vertical stress for a 10 meter stope span using different methods

5.3.7 Proposed Analytical Equation for Inclined Stopes

The following equation here derived applies to an inclined differential slice, thereby accounting for inclined stopes. The derivation itself can be found in Appendix A. Note that this expression can be considered an approximation only.

\[
\sigma_y(z) = \left( \frac{\gamma \cdot L}{2 \cdot K \cdot \tan(\phi)} \right) \cdot \sin^2(\beta) \cdot \left[ 1 - \exp\left( -\frac{2 \cdot K \cdot \tan(\phi) \cdot z}{L \cdot \sin^2(\beta)} \right) \right] \tag{5-2}
\]

In comparison to Blight’s analytical formulation for inclined stopes, equation 5-2 differs by the factor: sin (\(\beta\)). The following figure compares equation 5-2 with the maximum vertical stress that occurs employing numerical modeling for different stope dip angles.
Vertically Stress Comparison
Analytical vs. Numerical Modeling

Figure 5-16: Analytical vs. numerical modeling comparison for different stope dip angles

Results from the proposed analytical equation and from numerical modeling are consistent with each other.

5.4 Stability of Cemented Rockfill Sill Mats

5.4.1 Introduction

The stability of rockfill sill mats depends on two principal factors:
- sill mat strength; and
- the weight of backfill.

The next section considers these two factors in detail.

Figure 5-17 portrays stope geometry, strength, stress components, and failure modes in a sill mat pillar.
5.4.2  Sill Mat Strength – Friction and Cohesion ($\phi_c$)

The strength of rockfill material can be substantially improved by the addition of a binding material. The most evident improvement (between an uncemented fill and a fill treated with cement) is in shear and tensile strength. Cement bonds that form between fill particles introduce a cohesive component to the fill’s shear and tensile strength, and it is this cohesive component that is absent in an uncemented fill.

The following two soil components directly determine backfill strength:

- frictional forces, proportional to the internal angle of friction ($\phi$) resulting from interlocking solid particles; and
- Portland cement (or similar soil binder) which significantly increases actual fill cohesion by binding the solid particles to each other.

The effects of grain interlocking, and hence the magnitude of the internal friction angle, depend upon grain shape, overall particle size, and packing density (Thomas, et al., 1979).

The elastic modulus of backfill increases with the addition of a binding agent. Backfill typically has a low modulus of elasticity of approximately 1000 MPa to 100 MPa, and is normally one to two orders of magnitude less stiff than the surrounding rock.
5.4.3 Backfill Load

The load of backfill exerted on the top of the CRF sill mat was analyzed in detail in the previous section. This load is dependent upon the backfill properties: density and friction angle; and the geometry of the stope which includes the height of backfill material, span of the stope, and stope dip.

5.4.4 Proposed Rotational Analytical Formulation

The original rotational analytical formulation was derived for pastefill. At the Musselwhite mine (rockfill), the shearing resistance of a given hangingwall contact surface may not be particularly low, as would be expected for pastefill. Also, the three sill mats in existence at Musselwhite do not have low dip angles.

For these reasons, the following analytical formulation incorporates shearing resistance at the hangingwall contact surface. This case does not yield an especially conservative result (in shearing resistance). Safety considerations demand conservative estimation so that, in this particular case, only some extent of the shearing resistance should be taken into account, whereas full shearing resistance is indicated in figure 5-18. Additionally, consistent with this proposition, the research developed by Dirige, De Souza, and Chew (2001) implied that centrifuge modeling studies of pastefill sill mats indicate that, in stopes with smooth rock wall conditions, sill mat failure is caused by the fill self-weight, i.e., failure has little dependency upon fill binder content. Furthermore, Dirige, De Souza, and Chew concluded that, in stopes with rough rock wall conditions, the wall roughness itself contributes significantly to the stability of the sill mat during undercut mining. The present study implies that backfill sill mats can be designed with a high degree of stability using standard centrifuge physical modeling in combination with numerical modeling. This approach suggests that, for the sill mat rotational failure mode, shear strength should be considered a component of the analytical solution derived by Mitchell (1991).

A multiplier factor, \( \alpha \), ranging from 0 to 1, is added to the equation, corresponding to the estimated contact length at the hangingwall between the rock wall and the cemented rockfill material. As an alternative, the contact quality itself may be estimated to arrive at \( \alpha \). After visual inspection, if it is determined that there is poor quality contact between the rock wall
and the CRF, then an \( \alpha \) equal to zero should be used, yielding a result similar to that given by equation 3-36. Alternatively, estimation of the \( \alpha \) factor could be made by measuring the horizontal load exerted by the CRF material on the hangingwall/sill mat contact (see Chapter 8: Future Work), which would be an indirect measurement of the quality of the contact.

The following equation can still be considered conservative given that sill mat failure (when a circular shear failure occurs) produces additional shearing resistance as the pillar slides against the rock wall (\( \tau_i \)), as portrayed in figure 5-19.

\[
(\sigma_v + d \cdot \gamma) > \frac{\sigma_i \cdot d^2 + \alpha \cdot 2 \cdot \tau_i \cdot d \cdot L \cdot \sin^2(\beta)}{L \cdot (L - d \cdot \cot(\beta)) \cdot \sin^2(\beta)}
\]

Figure 5-18: Rotational failure considering shear strength in the hangingwall of the sill mat

Also, tensile failure could occur at an angle differing from \( \beta \) and a location differing from point “O”, as predicted by numerical modeling (discussed later). If tensile failure does occur at point “O” (figure 5-18), then only the failing mass (portrayed by geometric figure \( \mu \)) should be considered and not the total mass of the sill mat, as implied by Mitchell’s simplification. Doing so thus accounts for less weight than the value denoted by \( w \). Also, the vertical stress \( \sigma_v \) should be measured near the footwall, not near the center, as in Mitchell (1991).
The following figure (5-20) displays factors of safety for the different failure modes incorporating Musselwhite’s sill mat properties, vertical stress, and stope geometry. The minimum safety factor required is 1.2 in the design of non-entry methods, and 1.5 for entry methods (Pakalnis, course notes, 2004).
5.5  **Failure Modes - Numerical Modeling Solution**

A series of models is run to obtain design curves representing sill mat failure modes. These parametric studies account for variation in sill mat strength via variations in cohesion, and via stope geometry (characterized by span, and dip angle) variations. In all cases, in the interest of time efficiency of the numerical modeling analysis, a constant vertical stress of 0.41 MPa (in the case of a vertical stope of 10 meter span and 35 meter rockfill height) acting along the entire stope span is applied to the sill mat pillar. As indicated previously, the distribution of the vertical stress along the stope span is *not* constant, and decreases with stope dip. However, this assumption (using a constant of 0.41 MPa) yields an extra factor of safety in the sill mat design.

5.5.1  **Model Construction**

The following sections deal with a non-standard grid of CRF sill mat pillar stability. There is no conventional standard for a grid portraying measurements of stability in CRF sill mat pillars, due to the fact that the pillar dimensions (height and span) vary according to strength properties assigned to the grid. These pillar dimensions result from a trial and error execution of the model with given strength properties.

An important consideration of the models is the interface elements designed to simulate distinct planes along which slip and/or separation can occur. More about interface elements can be found in Appendix C.

Once the grid is constructed, material properties must be assigned to the individual grid elements. Elastic host rock properties are assigned to the outer grid elements. *In situ* host rock properties are obtained form intact rock cores (i.e., in laboratory testing) and results are subsequently scaled to the *in situ* reality. For details of host rock properties, refer to Chapter 4: Database.

For the elastic model, the relevant properties are: density; bulk modulus; and shear modulus.

Even though backfill properties could be obtained from existing literature, it is the implication(s) of varying these parameters that is the principal objective here. These implications are discussed in detail in the next section.
The following table (5-1) portrays an example of Musselwhite’s sill mat geometry, rockfill loading, and strength properties.

**Table 5-1: Example of Musselwhite’s geometry, rockfill loading, and sill mat strength properties**

<table>
<thead>
<tr>
<th>Sill Mat Geometry</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Span, L</td>
<td>10.0 m (variable)</td>
<td></td>
</tr>
<tr>
<td>Height of Sill, d</td>
<td>7.0 m (variable)</td>
<td></td>
</tr>
<tr>
<td>Stope Dip, β</td>
<td>85° (variable)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rockfill Loading</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Stress, σv</td>
<td>0.41 MPa (constant)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sill Mat Strength Properties</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined Compressive Strength Test, UCS</td>
<td>8.0 MPa (variable)</td>
<td></td>
</tr>
<tr>
<td>Cohesion, c</td>
<td>2.0 MPa (variable)</td>
<td></td>
</tr>
<tr>
<td>Tensile Strength, σt</td>
<td>0.8 MPa (variable)</td>
<td></td>
</tr>
<tr>
<td>Shear Strength, τs</td>
<td>2.1 MPa (variable)</td>
<td></td>
</tr>
<tr>
<td>Friction angle, φ</td>
<td>38° (constant)</td>
<td></td>
</tr>
<tr>
<td>Density, δ</td>
<td>2.0 t/m³ (constant)</td>
<td></td>
</tr>
</tbody>
</table>

### 5.5.2 Constitutive Equations – Strain Softening

Swan and Brummer (2001) analyzed a strain softening model where a cohesive sill will lose integrity at a plastic strain of 1.5%. The CRF sill mat properties assigned by this thesis are of a Mohr-Coulomb type of material with strain-softening behavior that loses its cohesion, and tensile strength, from the given value to zero (MPa, psi, etc.), at a plastic strain of 1.5%. Correspondingly, the friction angle falls from the original value of 37 degrees (at a plastic strain of 0%) to 15 degrees (at a plastic strain of 1.5%). At a plastic strain of 1.5%, cohesion will be zero but there is still some strength because of the friction angle. For the Mohr-Coulomb plasticity model, the relevant properties are density, bulk modulus, shear modulus, friction angle, cohesion, dilation angle, and tensile strength. Figure 5-21 illustrates an example of a strain-softening model for the cohesion strength parameter.
5.5.3 Model Execution

For a given value of cohesion, span, and dip of the stope, the maximum height of the CRF sill pillar that results in failure can be determined by trial and error. Setting the actual stable height 25 centimeters above the height determined, results in a factor of safety of at least 1.0. In this case, 25 centimeters is arbitrarily determined, yet this choice represents an efficient compromise between the accuracy of a low value (time consuming) and the time saved in using a high value (which yields inaccuracy). Note that, for flexural, sliding, and rotational failure modes, the same model is applied where a specific failure mode results, according to the strength parameters: cohesion, tensile strength, and friction angle. For these three failure modes, two plots are presented: first, a plot of the grid elements and the applied vertical load, and second, a plot that illustrates the plasticity state of the grid elements.

Since the parameters are purposely set low enough to result in failure, the equilibrium solution is clearly not the objective here. When failure does occur, the model does not converge to the equilibrium state. Rather, the model is stepped through the simulation process, plotting the resulting collapse as it occurs. The minimum safety factor is thus achieved (≥1.0).
5.5.4 Caving Failure

Even though caving failure is the least probable failure mode for a sill mat of given strength, this mode is analyzed here theoretically, as if the strength path is dependent on the shape of the fracture caused by a caving failure. No difference between planar and semi-circular fractures can be found. Each caved pillar represents the same overall weight, but the difference is in the length of contact over the distance of the fracture.

Figure 5-22(a) portrays a planar fracture and figure 5-22(b) portrays a semi-circular fracture. From this analysis, no conclusions can be drawn regarding path dependence. It is apparent that both failures yield the same results.

![Figure 5-22: Caving failure mode – planar and semi-circular crack](image)

5.5.5 Flexural Failure

Figure 5-23(a) and figure 5-23(b) portray a thin sill pillar failing as a result of flexing. It is evident that the top grid elements fail in compression, and that tension cracks form on the floor of the pillar, subsequently causing its collapse. Plasticity indicators (figure 5-23(b)) are included. These identify the type of failure, e.g., shear, i.e., compressive failure, (represented by black in figures 5-23(b), 5-24(b), and 5-25(b)) or tensile failure (represented by white in figures 5-23(b), 5-24(b), and 5-25(b)), and whether the stress state in the zone is currently at the yield surface (“at yield”), or has previously reached the yield surface but is currently
below the yield surface ("at yield in past"). (Note: shaded area represents elements that do not fail.)

5.5.6 Sliding Failure

The following figures illustrate shearing failure of hangingwall and footwall sill-host rock contact. These figures represent a typical shearing failure case (out of many possibilities).

Figure 5-24: Sill mat sliding failure mode – grid elements and plasticity state
5.5.7 Rotational – Crushing Failure

Rotational-crushing failure mode is not conceived as presented here, in the analytical solution previously discussed. However, rotational-crushing failure modeling illustrates that failure occurs when the sill in contact with the hangingwall crushes (i.e., fails in compression) as the sill mat rotates. Tensile failure of the footwall/sill contact results in the rotation of the sill with respect to the bottom footwall/sill contact. These failures must occur simultaneously in both the hangingwall and the footwall.

Figure 5-25: Sill mat rotational crushing failure mode – grid elements and plasticity state

5.5.8 Rotational – Breaking Failure

A similar result occurs in the rotational-breaking failure mode, where the centrifuge model (Mitchell, 1991) predicts that a tensile failure propagates at the same angle as the dip of the stope, rotating about point “O”. Numerical modeling predicts that this failure occurs at approximately mid-span, with no particular angle of failure. The numerical solution implies that failure occurs where the sill, in contact with the hangingwall, crushes (i.e., fails in compression), as the sill mat rotates and breaks.
Figure 5-26: Sill mat rotational breaking failure mode – grid elements and plasticity state

5.5.9 Sill Mat Design Curves

Figures 5-27 to 5-38 represent a series of design curves that predict, in each case, the point of failure and the failure mode, for a given strength and specific geometric parameters. In each graph, the y-axis portrays (in meters) the minimum height, or depth, of a CRF sill mat necessary to be in a stable state. The graphs are of three types: no strength on hangingwall contact, 50% strength, and 100% strength on hangingwall contact, as corresponding to the percentage of the CRF cohesion value. E.g., for an 85 degree stope, with 10 meter span, when hangingwall/sill contact equals 0.75 MPa, i.e., 50% of full cohesion (1.50 MPa), rotational breaking occurs below 2.5 meters (see figure 5-31). Each individual graph represents a specific stope dip angle, ranging from 90 degrees (vertical) to 75 degrees. The individual curves within each graph represent specific stope spans, ranging from 6 meters to 12 meters. The type of failure occurring (below the minimum sill mat vertical height necessary to be stable) is indicated for each curve by the following notations: “S” for sliding, “Re” for rotational-crushing, “Rb” for rotational-breaking, and “F” for flexural, failure modes. All cases assume uniform vertical stress acting on the top of the CRF sill mat. A stable CRF sill mat is represented as being anywhere above the individual design curve in question (e.g., 6, 8, 10, or 12 meter span), implying a safety factor of 1.0 or greater.
Figure 5-27: Sill mat stability for 90° – No strength on HW (τᵢ = 0%)

Figure 5-28: Sill mat stability for 90° – 50% sill mat strength on HW (τᵢ = 50%)

Figure 5-29: Sill mat stability for 90° – 100% sill mat strength on HW (τᵢ = 100%)
Figure 5-30: Sill mat stability for 85° – No strength on HW ($\tau_t = 0\%$)

Figure 5-31: Sill mat stability for 85° – 50% sill mat strength on HW ($\tau_t = 50\%$)

Figure 5-32: Sill mat stability for 85° – 100% sill mat strength on HW ($\tau_t = 100\%$)
Chapter 5 – Design Guidelines for Cemented Rockfill Sill Mats

Figure 5-33: Sill mat stability for 80° – No strength on HW (τ_1 = 0%)

Figure 5-34: Sill mat stability for 80° – 50% sill mat strength on HW (τ_1 = 50%)

Figure 5-35: Sill mat stability for 80° – 100% sill mat strength on HW (τ_1 = 100%)
Figure 5-36: Sill mat stability for 75° – No strength on HW ($\tau_t = 0\%$)

Figure 5-37: Sill mat stability for 75° – 50% sill mat strength on HW ($\tau_t = 50\%$)

Figure 5-38: Sill mat stability for 75° – 100% sill mat strength on HW ($\tau_t = 100\%$)
This series of design curves enables the operator to select the sill dimensions, given the cohesive strength of the fill material, and the parameters considered of greatest significance to the design such as:

- span;
- wall dip; and
- HW shear strength.

The Flac$^{2D}$ approach used in this study does not incorporate a factor of safety in its method of design. However, by applying a factor of safety to the cohesion parameter, the total factor of safety in the overall design will be improved.
6 EFFECT OF DELAYED BACKFILL ON OPEN STOPING

6.1 Introduction

A significant question at the Musselwhite mine and for open stope operators in general, is what is the effect of mining adjacent to previously backfilled stopes? Design methods such as The Stability Graph have been derived from a database of isolated stopes. These are stopes that have solid abutments adjacent to the stope being mined.

The Stability Graph Method makes the assumption that the adjacent backfilled stope does not reduce overall stability.

This thesis, alternatively, investigates the effect of backfill on adjacent stopes in terms of resultant wall slough.

6.2 Backfill as Local Support

Backfill should generate induced stresses on the order of 30 MPa, to provide for significant local support (Gürtunca et al., 2001). However, the research by Gürtunca implies that backfill affects displacement and stresses within the rock mass at a much lower backfill stress level. This research indicates, in terms of backfill providing local support, that backfill starts to induce changes within the rock mass when backfill stresses approach 1 MPa and furthermore, that rock mass induced effects reach their maximum when backfill stresses approach 2 - 3 MPa. These effects are measured via reduced wall closure, reduced bed separation, reduced dilation, and the effect of induced wall stresses.

Reduced closure and dilation within the rock mass limits stope wall disturbance/conditioning, and in turn lowers wall slough. This level of disturbance/conditioning is a function of distance to the backfill front. Backfill strain at a specific distance near the open stope/fill face should yield (a critical) 2 - 3 MPa of induced backfill stress, in theory. If this critical 2 - 3 MPa backfill stress occurs, but a distance much greater than that of the exposed strike length of the stope, then clearly these conditions do not adequately influence the exposed area, and do not render stability within the rock mass. If the selected mine layout does not include within its parameters sufficient closure, then the use of backfill in such a low closure environment is not fully effective and consequently provides negligible support to the adjacent stope.
Consequently, it is evident that, when mining adjacent to backfill (e.g., Avoca and secondary transverse open stoping), the radius factor (or hydraulic radius) of the exposed area is only one of several geometric factors determining stability in open stopes. A combination of the radius factor (or hydraulic radius) of the exposed zone and the backfilled zone should be used to account for the stability of open stopes.

For mining via the Avoca method utilizing unconsolidated rockfill, or in the case of transverse open stope mining with cemented rockfill primaries, the main consideration is the effect of mining adjacent to backfill. Taking into account that, at the actual depth of mining (200 - 600 meters below the surface), wall closure is negligible, and that rockfill will generate minimum horizontal stresses, less than the minimum threshold of 1 MPa is generated, i.e., not enough to provide local support. As a result, it is necessary to adjust the stability graph curves (or modify the radius factor) in accordance with this minimum support condition. Radius factor (or hydraulic radius) must be increased in order to account for the backfilled zone and its effects on the overall exposed surface.

Figure 6-1 displays the horizontal stresses exerted by the backfill in a 35 meter high stope (unconsolidated rockfill) at different stope spans. It is evident that the horizontal stress values are very low and are far below that needed to provide local support (2 - 3 MPa of horizontal stress).

Figure 6-1: Horizontal stress versus rockfill height at different stope spans
6.3 **Use of the Stability Graph Method**

All of the known Empirical Design Stability Methods (re: Chapter 3) were developed for isolated stopes and *not* for stopes mined adjacent to backfill (Figure 6-2). This leads to fundamental errors if these existing methods are not calibrated to fit the given (non-isolated) specific mining conditions (e.g., mining adjacent to backfill). The calibration of the stability graph, the modified stability graph, or the hangingwall stability rating graph (re: Chapter 3), when mining adjacent to a backfilled stope, is accomplished by modifying the radius factor (or hydraulic radius) thereby increasing its value, and then applying the method to the circumstance of mining adjacent to a backfilled stope. Interpolation (rather than extrapolation) is appropriate to empirical methods. New data points thus enhance the existing database. Numerically modeling the significance of mining adjacent to backfill will improve our understanding of stope stability.

*Figure 6-2: Stope adjacent to rock (isolated) or backfill abutment(s) – Modified from Atlas Copco Drawing*
6.4 Quantifying the Effect of Backfill on Longhole Open Stoping

Two mining methods are employed at Musselwhite: Avoca (or longitudinal retreat), and transverse open stoping. Effects of mining adjacent to backfill are assessed for isolated stopes, and for stopes mined adjacent to backfill, via the following procedure:

1. Compilation of the Musselwhite database in terms of hydraulic radius (converted to radius factor) and the modified stability number $N'$;
2. Analysis of other parameters that affect the stability of open stopes such as undercutting of stope walls;
3. Conducting structural mapping within the different rock units;
4. Determining Equivalent Linear Overbreak Slough (ELOS) for the compiled database using CMS survey profiles;
5. Establishing, using neural networks, the relative importance of individual parameters and their significance on wall stability (ELOS) for isolated stopes versus stopes mined adjacent to backfill;
6. Determining via numerical modeling (finite difference method), the resultant decrease on wall closure for stopes with and without backfill;
7. Determining wall closure using three dimensional numerical modeling (3D boundary element method) in order to assess the effect of wall closure as further mining of the stope progresses; and
8. Determining the change in radius factor throughout the stope wall as mining progresses, i.e., away from the original radius factor reference point.

6.5 Updating Musselwhite’s Stability Graph

Musselwhite’s stability database is plotted on the Stability Graph, in terms of radius factor, with the equivalent linear overbreak curves superimposed onto it. Figure 6-3 portrays this updated Stability Graph for Avoca (or longitudinal retreat) stope data, and figure 6-4 portrays the updated Stability Graph for transverse stope data. 90 and 48 points are plotted for longitudinal and transverse stopes respectively, for a total of 138 points, all of which are plotted in figure 6-5.
Empirical Estimation of Overbreak/Slough
Musselwhite Mine (Longitudinal Stopes Data) 90 cases

Figure 6-3: Stability Graph update – longitudinal stope data (AVOCA)
Figure 6-4: Stability Graph update – transverse stope data

The data points of the updated database are summarized in the following table:

Table 6-1: Average and standard deviation for the Musselwhite’s stability database

<table>
<thead>
<tr>
<th></th>
<th>Longitudinal stopes</th>
<th>Transverse stopes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average</td>
<td>Std Deviation</td>
</tr>
<tr>
<td>Length FW</td>
<td>42.0</td>
<td>20.8</td>
</tr>
<tr>
<td>Length HW</td>
<td>41.4</td>
<td>20.0</td>
</tr>
<tr>
<td>Height FW</td>
<td>33.9</td>
<td>8.0</td>
</tr>
<tr>
<td>Height HW</td>
<td>34.3</td>
<td>7.6</td>
</tr>
<tr>
<td>Dip FW</td>
<td>92.4</td>
<td>8.7</td>
</tr>
<tr>
<td>Dip HW</td>
<td>82.9</td>
<td>10.6</td>
</tr>
<tr>
<td>Hydraulic Radius FW</td>
<td>8.9</td>
<td>2.5</td>
</tr>
<tr>
<td>Hydraulic Radius HW</td>
<td>8.9</td>
<td>2.4</td>
</tr>
<tr>
<td>Radius Factor FW</td>
<td>9.8</td>
<td>2.7</td>
</tr>
<tr>
<td>Radius Factor HW</td>
<td>9.8</td>
<td>2.6</td>
</tr>
<tr>
<td>ELOS FW</td>
<td>0.5</td>
<td>0.4</td>
</tr>
<tr>
<td>ELOS HW</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Empirical Estimation of Overbreak/Slough
Musselwhite Mine (All Stopes Data) 138 cases

0.3m ELOS
1.0m ELOS
2.0m ELOS

Legend
- ELOS <= 0.5m
- 0.5m < ELOS <= 1.0m
- 1.0m < ELOS <= 1.5m
- 1.5m < ELOS <= 2.0m
- 2.0m < ELOS

Figure 6-5: Stability Graph update – transverse and longitudinal stope data
6.6 Neural Network Training and Results

Neural networks are used to determine the relative importance of the various inputs related to the stability of open stopes, and to determine the effect of backfill on open stope stability where mining adjacent to backfill occurs. The significance of individual inputs is assessed in terms of its effect on wall slough (ELOS).

6.6.1 Relative Importance of Inputs

Figure 6-6 implies that the dip of the orebody highly affects stope stability when dip angle is greater than 90 degrees (geometric footwall). From a geometric perspective, this is the point at which a hangingwall becomes a footwall.

![Relative Importance of Inputs](image)

**Figure 6-6: Relative importance of inputs on stope stability for stopes dipping more than 90 degrees (footwalls)**

Conversely, at orebody dip angle of less than 90 degrees, the quality of the rock mass is the principle determining stability parameter. Figure 6-7 portrays the relative importance of the parameters that determine stope stability.
Figure 6-7: Relative importance of inputs on stope stability for stope dips under 90 degrees (hangingwalls)

The parameters known to determine stope stability are (in no particular order): drilling and blasting, wall undercut, orebody dip, hydraulic radius of the stope walls, quality of the rock mass, stress state, geological heterogeneity, and level of support, among other less significant factors.

Current tools in use for the design of open stopes oversimplify all these parameters as being represented by quality of rock mass, $N'$, and the distance to the abutments (hydraulic radius). The assumption is that the quality of the rock mass subtends all parameters excluding hydraulic radius.
Figure 6-8 portrays the database of measurements gathered by Musselwhite’s engineering staff and incorporates other factors that influence stope stability. One significant factor is the thickness of the 4f rock, whose RMR value can be as low as 45%. Another variable determining stope stability, when combined with rock quality, is the degree of undercutting of the stope walls.

The quality of 4f rock usually determines the stability of the stope. Exposing the 4f rock may produce caving, especially when this rock type has been undercut. This is due to the foliated nature of 4f rock and its structure, where, typically, the joint sets with low strength sericitic-alteration-infilling, run parallel to the orebody.

![Relative Importance of Inputs](image)

**Figure 6-8: Relative importance of inputs on stope stability considering rock type, ore dip and undercut**

Figure 6-9 illustrates how numerical modeling can be used to determine stability in the hangingwall where the (poor-quality) rock mass is undercut. The modeling results imply that when this poor-quality rock type is not undercut, the stope walls are stable. Naturally, this behavior may vary significantly in a real environment, due to the numerous variables involved in a real-world situation. Some elements of these real-world variables are difficult, if not impossible, to represent by numerical modeling.
6.6.2 Neural Network Predictions

The "neural net" analysis is trained on two databases, independent of each other:
- Isolated stopes (transverse primaries stopes); and
- Stopes adjacent to backfill (Avoca).

This analysis is employed to determine if any relationship exists between ELOS values when mining adjacent to backfilled stopes versus mining adjacent to solid abutments. ELOS predictions are made for various hydraulic radius and stability values. The trained results generated for longitudinal stopes are expected to be higher in terms of ELOS than results generated for transverse primary stopes, but no such difference can be found where backfill is used in actual practice, even though radius factor measurements should be somewhat higher (re: Section 6.7).

Figure 6-10 portrays the predicted ELOS values for transverse primaries versus longitudinal stopes.
Figure 6-10: Predicted ELOS values of transverse vs. longitudinal stopes

Figure 6-10 illustrates the random scatter generated from neural net analysis, as opposed to the expected distribution of predicted data which should have been located primarily below the one-to-one regression. It is not clear that longitudinal stopes (adjacent to backfill) would yield higher ELOS values than would transverse primary stopes (i.e., isolated), as expected. This result can be justified by the many other factors that affect stability such as drilling and blasting, geological heterogeneity, undercutting of the stope wall, and others. An example of unexpected wall slough is portrayed by figure 6-11, where a CMS survey of a section of the 500SZN-P1 zone shows that dilution occurs due to a geological structure.
Figure 6-11: Wall instability due to structure on hangingwall
6.7 Effect of Backfill in Stability of Open Stopes – Numerical Modeling Analysis

The following sequences are tested using Flac$^{2D}$ to quantify the effect of mining adjacent to backfill. The first sequence (figure 6-12), corresponds to the excavation of a 40 meter high bottom stope and subsequent excavation of the top stope (of the same height) without using backfill for the bottom stope. The next sequence (figure 6-13) is the same as the first but in this case, after the bottom stope has been mined, it is then backfilled. The third sequence (figure 6-14) considers only mining of the top stope.

Figure 6-12: Sequence without backfill

Figure 6-13: Sequence with backfill
Figure 6-14: Mining of the top stope only

Figure 6-15 implies that backfill has a minimal effect when compared with solid rock abutments. These results are based solely on an elastic, numerical model. In practice, this is not the case, as backfill provides confinement, in terms of stability. The assumption that it is indeed a minor effect on elastic wall closure explains why there is an increase in radius factor (or hydraulic radius) when mining adjacent to backfill. It is important to note that this is a parametric study, measuring the effect of backfill in an elastic, homogeneous, isotropic model.

Figure 6-15: Horizontal displacement profile
6.7.1 Radius Factor Behavior with Stope Strike Length

Radius factor is calculated for various points from the front face, while increasing the stope strike length until the radius factor reaches a constant value, beyond a specific strike length. Beyond this strike length, radius factor variation is minimal and begins to behave asymptotically. The assumption being made here is that, for a given exposed strike length, the measuring point is one-half of the maximum unsupported span, and beyond that maximum unsupported span the stope is considered to be backfill. Figure 6-16 portrays a measurement point at 5 meters from the front face, where it can be assumed that the maximum unsupported strike length is 10 meters, beyond which the stope is considered to be backfilled. At 5 meters from the front face, the radius factor is calculated for a stope of 10, 20, 30, 40, 50, 100, 200, and 400 meter strike lengths.

![Diagram of stope with backfill behavior](image)

Figure 6-16: Schematic of a stope of increasingly larger strike length
Figure 6-17 illustrates a radius factor measurement at 5 meters from the front face, for a stope of 40 meter height and 100 meter strike length. The same procedure is used to measure the radius factor at 6, 7, 8, 9, 10, 15, and 20 meters from the front face, representing 12, 14, 16, 18, 20, 30, and 40 meters, respectively, of unsupported strike length. Beyond these distances (12, 14, 16, etc.) the stope is considered to be backfill (figure 6-17 portrays the 5 meter case only).

Figure 6-18 illustrates the generation of radius factors by an AutoLISP program (Milne and Lunder, 1994) over an entire area of a stope of 40 meter height and 100 meter length.
Figure 6-18: Radius factor generation

Figure 6-19 portrays the relative increment of radius factor at increasingly larger stope strike length measured at a distance from the front face. Radius factor behaves asymptotically with the stope strike length.

Figure 6-19: Radius factor versus stope strike length at different distance from front face
6.7.2 Horizontal Displacement Behavior considered with Stope Strike Length

The procedure is repeated but using the Map3D boundary element program in order to measure horizontal wall closure at 5, 6, 7, 8, 9, 10, 15 and 20 meters from the front face. These measurements represent 10, 12, 14, 16, 18, 20, 30 and 40 meters respectively of maximum unsupported strike length. As before, the assumption is that, beyond these distances, in each case, the stope can be considered as a backfilled stope. There is a decreasingly minor (asymptotic) effect on wall closure. Figure 6-20 portrays horizontal wall closure measurement at 5 meters from the front face for a stope of 40 meter height and 100 meter strike length.

Figure 6-20: Horizontal displacement measurement at various distances from front face

Figure 6-21 illustrates the relative incremental decrease in wall closure at progressively larger stope strike lengths. From this analysis it is evident that radius factor is correlated with horizontal wall closure. Wall closure behaves asymptotically with stope strike length. The increase in wall displacement is proportional to the degree of failure, which itself is limited in degree (i.e., displacement) by the distance to the abutments.
Chapter 6 - Effect of Backfill on Open Stoping

No significant effect of backfill on wall displacement was found from an elastic, homogeneous, isotropic, numerical modeling perspective. This parametric assessment analyzes the effect of backfill only, and does not incorporate other factors such as blasting damage, rock heterogeneity, etc. The model is an approximation only.

However, this model predicts, that greater displacement leads to a proportional increase in the degree of failure. Backfill alone does not affect the degree of failure in a meaningful way. The following conclusion is reached: radius factor for a stope mined adjacent to backfill should consider the exposed wall and at the same time consider the backfilled wall (in terms of its effect on stability).

6.7.3 Design Curves – Avoca Mining Method

Figures 6-22 and 6-24 illustrate the increase of radius factor from its initial value (represented by the isolated stope curve) to its maximum value (represented by the 380 meter backfill curve). Figures 6-22 and 6-24 portray radius factors for stopes of 30 meter and 40

Figure 6-21: Horizontal displacement versus stope strike length at different distances from front face

No significant effect of backfill on wall displacement was found from an elastic, homogeneous, isotropic, numerical modeling perspective. This parametric assessment analyzes the effect of backfill only, and does not incorporate other factors such as blasting damage, rock heterogeneity, etc. The model is an approximation only.

However, this model predicts, that greater displacement leads to a proportional increase in the degree of failure. Backfill alone does not affect the degree of failure in a meaningful way. The following conclusion is reached: radius factor for a stope mined adjacent to backfill should consider the exposed wall and at the same time consider the backfilled wall (in terms of its effect on stability).

6.7.3 Design Curves – Avoca Mining Method

Figures 6-22 and 6-24 illustrate the increase of radius factor from its initial value (represented by the isolated stope curve) to its maximum value (represented by the 380 meter backfill curve). Figures 6-22 and 6-24 portray radius factors for stopes of 30 meter and 40
meter heights, respectively. The dashed line (---) represents the maximum radius factor obtained at the middle of the stope, i.e., 200 meters from each end.

Figure 6-22: Radius factor increment – stope of 30 meter height and 400 meter strike length
Example calculation: radius factor measurement is obtained for point R of the isolated stope ABCD (figure 6-23) and is also measured for the exposed stope plus the backfilled stope AEFD (figure 6-23) which can then in its entirety be assumed to be open. In the case of a 30 meter stope height, the radius factor increases from an initial value of 6.5 meters for a 20 meter strike length to a constant value of 8.4 meters at a 400 meter strike length, as illustrated by figure 6-22 and depicted three dimensionally in figure 6-23.

![Figure 6-23: Radius factor increments for a stope of 30 meter height and 400 meter strike length](image)

For a stope of 30 meter height, where radius factor is measured at a mid-span of 10 meters (20 meter strike length), radius factor increases from an initial value of 6.5 meters (in the isolated stope) and reaches the constant value of approximately 8.4 meters at 400 meter strike length (mid-span remains 10 meters) for 380 meters of backfill.
Chapter 6 – Effect of Backfill on Open Stoping

Figure 6-24: Radius factor increments for a stope of 40 meter height and 400 meter strike length

This approach shows that it is numerically possible to assess the increase in radius factor due to mining adjacent to backfill. After applying neural network analysis, the mine database (see Chapter 4, Table 4-7) does not conclusively reflect a quantitative procedure, as a consequence of the many parameters that affect dilution.
Chapter 7 - Conclusions and Recommendations

7 CONCLUSIONS AND RECOMMENDATIONS

Backfill plays an essential role in the efficiency of mining operations at the Musselwhite ore deposit. Monitoring the true performance of backfill, both from safety and operational perspectives, should be a routine part of operations to ensure that the quality of the mine’s final product results as planned.

Cement is the principle cost component of a backfill system. Strategies that optimize the use (i.e., amount) of cement, while maintaining a high degree of quality in backfill material, are of substantial benefit to the overall mining operation.

In order to determine analytically the vertical load exerted by unconsolidated backfill, the coefficient of lateral earth pressure, $K$, must be obtained. An inaccurate estimation of $K$ will in turn produce a sill mat design differing from the optimized solution. Numerical modeling was used to determine directly the coefficient $K$ by measuring the horizontal and vertical stresses of a given grid element, and subsequently calculating the ratio between these two stresses. The coefficient of lateral earth pressure $K$ obtained via numerical modeling falls between the active value and the at rest value. Additionally, a curve was fitted to the points obtained from this numerical modeling and an equation was subsequently determined. This equation can be used to obtain the $K$ value for unconsolidated rockfill material confined within stope walls.

The equation is:

$$K = 1.4 \cdot Sin^2(\phi) - 2 \cdot Sin(\phi) + 1$$

with $0^\circ < \phi \leq 40^\circ$

Also, an analytical equation to determine the vertical load of unconsolidated backfill as a function of backfill height was derived in this thesis. This equation accounts for the decrease in vertical load as a result of stope inclination. Although this equation yields an approximation, the numerical modeling conducted was shown to be highly correlated with analytical results.

The equation is:
The existing analytical equation for the rotational failure mode does not consider shear strength on hangingwall/sill mat contact, due to the low strength of pastefill and/or the gap that may occur at the contact surface (as a result of low dip angle), given the method of pastefill delivery itself (i.e., via piping), and so it becomes difficult to maintain firm contact. Methods of constructing sill mat pillars employing rockfill most likely can overcome this problem, in nearly all cases, and so a new analytical equation that considers shear strength of the wall contact was developed. As a result of this new analytical equation, there is a significant increase in the factor of safety of a given sill mat.

The equation is:

\[
\sigma_v(z) = \left( \frac{\gamma \cdot L}{2 \cdot K \cdot \tan(\phi)} \right) \cdot \sin^2(\beta) \cdot \left[ 1 - \exp \left( -\frac{2 \cdot K \cdot \tan(\phi) \cdot z}{L \cdot \sin^2(\beta)} \right) \right]
\]

Although analytical equations are highly useful in the design of sill mat pillars, this thesis proposed that numerical modeling is a more accurate method of designing sill mat pillars given numerical modeling’s capacity of incorporating several additional factors that influence stability. These factors include: material deformation, and the reduction in strength properties throughout the modeling process (as a function of the strain on grid elements). Among the strategies assessed, the finite difference program Flac²D was considered to be the most suitable program of all those available, in addressing and understanding the mechanics of the various failure modes of cemented rockfill sill mats, and the parameters involved in rockfill sill mat design.

Design curves for the stability of cemented rockfill sill mats were developed to aid in the design of new pillars or to reassess the performance of existing pillars at the Musselwhite mine. It is imperative that, when mining under backfill, no personnel are exposed to the danger, and remote mining methods are employed (with a minimum factor of safety of 1.2 for non-entry methods). This is the lowest useable factor of safety to ensure that the sill mat remains integral during its exposure. The above statement notwithstanding, via numerical
modeling it is possible only to determine sill mat pillar stability with a factor of safety of 1.0, and by incorporating a suitable factor of safety on the input strength parameters (cohesion) the operator is able to produce a design within existing construction guidelines. It is also essential that stability calculations dictating variation in fill recipes be confirmed by full-scale field trials and verified by observations during actual mining. If at all possible, numerical modeling is the preferred method of analysis (over analysis using analytical equations).

The graphs shown in detail in Chapter 5 portray the stability curves developed via numerical modeling. The x-axis represents the strength of the sill mat pillar (in terms of cohesion) and the y-axis represents the minimum required vertical height of the pillar to be stable. Each graph includes four stability curves for spans ranging from 6 to 12 meters. The graphs are divided into 3 categories: zero strength in the hangingwall/sill mat contact, 50% strength in the contact, and 100% strength assumed in the contact (cohesion value). Graphs were developed for 90 degrees (vertical), and for 85, 80 and 75 degree stopes.

The rotational failure mode obtained via numerical modeling differs from that obtained in previous existing research employing centrifuge models. Two rotational failure modes were generated via numerical modeling, rotational-crushing and rotational-breaking. Neither implied tensile failure at the footwall propagating at an angle of the same magnitude as the stope dip angle, but in a direction defined by the supplementary angle, as predicted by Mitchell’s (1991) rotational failure mode equation. Rotational-crushing failure mode predicts tensile failure at the footwall contact and shear (or volumetric) failure at the hangingwall contact, whereas the rotational-breaking failure mode predicts a tensile failure propagating vertically at mid-span, and a shear (or volumetric) failure at the hangingwall contact.

Musselwhite’s Stability Graph was updated using the cavity monitoring system survey database. A total of 138 cases were analyzed. These were divided into isolated stopes (48 cases) and stopes mined adjacent to backfill (90 cases). The points were plotted into the empirical estimation of wall slough (ELOS) expressed in terms of radius factor (portrayed in
detail in Chapter 6), since the adjustment factor was obtained only for the radius factor case (which would not be possible in the case of hydraulic radius).

Neural network analysis was implemented to determine if isolated stopes would account for less wall slough compared with stopes of the same exposed wall size, but where the mining occurs adjacent to backfill. As a consequence of the many variables involved (principally, drilling, blasting and geological heterogeneity), no conclusive results were obtained from this analysis in order that the assumption could be made that stopes mined adjacent to backfill will result in a greater degree of wall slough than isolated stopes.

The existing empirical design–stability methods were mostly developed for isolated stopes. Although it is possible to calibrate the empirical design stability curves to site specific conditions (e.g., mining adjacent backfilled stopes), the method proposed in this thesis uses the same design curves developed for isolated stopes, but adjusts the magnitude of the radius factor of the excavation to account for the portion of the stope containing backfill. The figures shown in detail in Chapter 6, portray the incremental increase in radius factor that must be progressively applied as mining advances, where mining adjacent to backfilled stopes occurs. The x-axis represents the distance to the front face. This thesis made the assumption that, for a given exposed strike length, the measuring point is one-half of the maximum unsupported span, and beyond that maximum unsupported span the stope is considered to be backfill. The y-axis displays the radius factor values. Two curves are plotted, the lower representing radius factor for isolated stopes, and the upper representing maximum radius factor obtained for a 380 meter backfilled stope (strike length) that can be considered infinite. Two graphs were developed, one for a stope of 30 meter height and the other for a stope of 40 meter height.

The increase in radius factor will vary depending upon exposed strike length. There is a negligible incremental difference in the radius factor value for a backfilled stope of 100 meter strike length compared to a backfilled stope of “infinite” strike length. A stope of 400 meter strike length is considered to be the same mathematically as a stope of infinite strike length.
A hypothetical effect should be that secondary stopes, where both abutments are CRF pillars, will have greater instability as consequence of higher radius factor, and in turn will suffer greater dilution (when compared to stopes with a single backfill abutment).

Radius factor calculations obtained in this thesis using the AutoLISP procedure (Milne and Lunder, 1994), always yield the maximum value for each location measured (since the stope is considered entirely without backfill). For this reason, the radius factor value obtained in each case is the maximum value attainable. Consider a reduced percentage of the radius factor maximum value as being effective in reducing wall closure, due to backfill’s minimal effects on wall closure as predicted by the numerical modeling results in this thesis.

The increased use of pastefill/CRF in mining operations demands a greater understanding of the geomechanical effects of backfill and their implications on design. This area of application will gain increasing acceptance in the future, as mining deposits are found deeper underground and ground conditions prove to be more hazardous.
8 FUTURE WORK

Implementing a full instrumentation campaign for determining in situ properties (i.e., strength, stress and strain) of cemented and uncemented rockfill is highly recommended, as initially proposed in the research by Scoble, et al. (1987). A better understanding of these properties will contribute to the optimization of CRF sill mat designs. Additionally, optimal design of stopes, in terms of excavation dimensions, leads to a reduction in wall instability, where mining adjacent to backfilled stopes occurs. An optimized design directly reduces mining costs.

Unconsolidated rockfill vertical loads along the span of a backfilled stope could be determined using an array of earth pressure cells installed on top of the sill mat pillar. In practice, this is a very difficult task due to the physical hazards of entering an open stope. Remoted equipment would thus be employed to install this equipment, minimizing the risk to mine personnel.

CRF sill mat strength properties could be determined via an in situ pressuremeter apparatus. Pressuremeters have cylindrical rubber membranes that are inflated with pressurized fluid (e.g., water, gas, or oil) to measure volume change. Entire stress-strain-strength curves could then be derived, as well as in situ total horizontal stress, shear modulus, shear strength, and limit pressure. Types of pressuremeter testing include: the pre-bored (Menard), the push-in device, the self-boring, and the full-displacement, types (e.g., cone pressuremeter or pressiocone). Pre-bored types are preferred considering the extreme difficulty of penetrating cemented rockfill.

The strength on the hangingwall/sill mat contact could be indirectly inferred using earth pressure cells. The higher the load exerted by CRF backfill on the contact, the higher the inferred strength value. Horizontal stress, as well as the stress parallel to the hangingwall contact, should increase during mining of adjacent stopes (wall convergence). These load measurements would help in determining the crushing mode instability of sill mats (not analyzed in this research). Another possibility for determining the strength on hangingwall/sill mat contact is to obtain core samples through the contact and, using a shear
box apparatus, test the shear strength of the sample obtained. Samples should be cored at various depths of the contact and also at various points along the strike length of the sill mat pillar, in order to obtain a representative measurement of the strength of the hangingwall/sill mat pillar contact.

The effects on sill mat stability incorporating the actual vertical load distribution along the stope span, as well as, greater vertical height of rockfill, could be effected by numerical modeling analysis. In this thesis, the analyses were completed using a constant vertical load along the span.

A three dimensional numerical modeling program could be used to model backfill behavior. Three dimensional numerical modeling would help in that it can model the depth dimension, which cannot be modeled using the two dimensional version. The two dimensional version cannot accurately model the overall three dimensional geometry associated with AVOCA mining.

Also, numerical modeling to determine stope stability, where mining adjacent to backfill, could be re-run in three dimensions. The results may help determine how backfill affects displacement around an opening adjacent to the backfill itself. Instrumentation of the stope wall by employing extensometers could be used to determine the real displacements of both isolated stopes and of stopes mined adjacent to backfill.

Stopes with backfill below (longitudinal mining method with backfill on the floor of the stope) could be analyzed. The analysis performed in this thesis deals only with radius factor adjustments for stopes backfilled along the strike. Backfill on the floor of the stope would have a direct effect on the magnitude of the radius factor.

To determine solely the influence of backfill on wall stability, it would be necessary to isolate all external factors that account for wall instability. Those factors that can be isolated are mainly drilling and blasting. In order to isolate these factors, it would be necessary to survey all blastholes (or at least those blastholes drilled adjacent to the hangingwall and
footwall of the orebody). A direct measurement of blasting performance could be obtained via seismic wave method. Results from this analysis could be combined with drill deviation to account for unplanned wall slough as a result of dynamic effects. Doing so, would make it possible to determine the actual, real-world effect of backfill on wall stability.
REFERENCES


Sources


APPENDIX A

Proposed Analytical Equation to Determine Vertical Load of Backfill

This method is based on the Janssen’s method of silo theory, but considers inclined walls.

Differential inclined slice in a silo

This method considers the same characteristics of Janssen’s differential slice but incorporates the following assumptions: the horizontal pressure is equal in magnitude and in opposite direction on both silo walls; and the tangential stress is equal in magnitude for both sides of the silo walls. This will give the following “approximation” for the force equilibrium expression:

$$2 \cdot b \cdot \sigma_y - 2 \cdot b \cdot (\sigma_y + d\sigma_y) - \frac{2 \cdot dy \cdot \sigma_t}{\sin(\beta)} + 2 \cdot b \cdot \gamma \cdot dy = 0 \quad 1$$

Simplifying the above equation and dividing by $2b \cdot dy$ gives the differential form:

$$\frac{d\sigma_y}{dy} + \frac{\sigma_t}{b \cdot \sin(\beta)} - \gamma = 0 \quad 2$$
A further generalization of the above equation is obtained by defining the hydraulic radius of the silo for any cross section of any shape as \( HR = \frac{S}{P} \), where \( S \) is the area and \( P \) is the perimeter. With this notation, the equilibrium equation becomes:

\[
\frac{d\sigma_y}{dy} + \frac{\sigma_y}{HR \cdot \sin(\beta)} - \gamma = 0
\]

The friction law would be:

\[
\sigma_i = \sigma_n \cdot \tan(\phi)
\]

However, in the inclined case \( \sigma_n \) relates to \( \sigma_x \) with the angle of the wall \( \beta \), and then the friction law can be written as:

\[
\sigma_i = \frac{\sigma_x \cdot \tan(\phi)}{\sin^2(\beta)}
\]

The two unknowns, the average vertical stress \( \sigma_y \) and the average horizontal stress \( \sigma_x \) are necessary to solve equation 3. To obtain a solution, it is necessary to employ the coefficient of lateral earth pressure \( K \) previously defined.

\[
\sigma_i = \frac{K \cdot \tan(\phi) \cdot \sigma_y}{\sin^2(\beta)}
\]

Combining equations 3 and 6, yields:

\[
\frac{d\sigma_y}{dy} + \frac{K \cdot \tan(\phi) \cdot \sigma_y}{HR \cdot \sin^2(\beta)} - \gamma = 0
\]

The integrating factor would be:

\[
u(y) = \exp\left(\int \frac{K \cdot \tan(\phi)}{HR \cdot \sin^2(\beta)} dy\right) = \exp\left(\frac{K \cdot \tan(\phi) \cdot \gamma}{HR \cdot \sin^2(\beta)}\right)
\]
The solution of the first order differential equation would be:

\[
\sigma_y = \frac{\int \exp\left(\frac{K \cdot \tan(\phi) \cdot y}{HR \cdot \sin^2(\beta)} \right) \cdot dy}{\exp\left(\frac{K \cdot \tan(\phi) \cdot y}{HR \cdot \sin^2(\beta)} \right)}
\]

Solving the integral yields to the following equation:

\[
\sigma_y = \left[ \left(\frac{\gamma \cdot HR \cdot \sin^2(\beta)}{K \cdot \tan(\phi)} \right) \cdot \exp\left(\frac{K \cdot \tan(\phi) \cdot y}{HR \cdot \sin^2(\beta)} \right) + C \right] \cdot \exp\left(\frac{-K \cdot \tan(\phi) \cdot y}{HR \cdot \sin^2(\beta)} \right)
\]

Using the boundary conditions, the following solution is obtained:

\[
\sigma_y(z) = \left(\frac{\gamma \cdot HR}{K \cdot \tan(\phi)} \right) \cdot \sin^2(\beta) \cdot \left[1 - \exp\left(\frac{-K \cdot \tan(\phi) \cdot z}{HR \cdot \sin^2(\beta)} \right)\right]
\]

or,

\[
\sigma_y(z) = \sigma_{y, Max} \cdot \sin^2(\beta) \cdot \left[1 - \exp\left(\frac{-K \cdot \tan(\phi) \cdot z}{HR \cdot \sin^2(\beta)} \right)\right]
\]

Where:
- \(\beta\) = stope's dip

Now, when infinite stope strike length is considered:

\[
HR_{\text{lim}, SL \to \infty} = \frac{L \cdot SL}{2(L + SL)} = \frac{L}{2}
\]

Where:
- \(L\) = stope's span; and
- \(SL\) = stope's strike length.
Then equation 11 can be simplified to:

\[
\sigma_v(z) = \left( \frac{\gamma \cdot L}{2 \cdot K \cdot \tan(\phi)} \right) \cdot \sin^2(\beta) \cdot \left[ 1 - \exp\left( - \frac{2 \cdot K \cdot \tan(\phi) \cdot z}{L \cdot \sin^2(\beta)} \right) \right]
\]
Appendix B – Analytical Equations to Determine Sill Mat Failure Modes

APPENDIX B

Proposed Analytical Equation to Determine Rotational Sill Mat Failure Modes

Driving momentum with respect to point O is:

\[ \sigma_v \cdot L \cdot \left( \frac{L}{2} - \frac{d}{2 \cdot \tan(\beta)} \right) + d \cdot L \cdot \gamma \cdot \left( \frac{L}{2} - \frac{d}{2 \cdot \tan(\beta)} \right) \tag{1} \]

Whereas resisting momentum with respect to O is:

\[ \sigma_i \cdot \left( \frac{d}{\sin(\beta)} \right) \cdot \left( \frac{d}{2 \cdot \sin(\beta)} \right) + \alpha \cdot \tau_i \cdot \left( \frac{d}{\sin(\beta)} \right) \cdot L \cdot \sin(\beta) \tag{2} \]

Then, rotational failure would develop when:

\[ \frac{\sigma_i \cdot \frac{1}{2} \cdot \left( \frac{d^2}{\sin^2(\beta)} \right) + \alpha \cdot \tau_i \cdot d \cdot L}{L \cdot \frac{1}{2} \left( L - \frac{d}{\tan(\beta)} \right)} > (\sigma_v + d \cdot \gamma) \tag{3} \]

Simplifying equation 3 yields:

\[ \frac{\sigma_i \cdot \left( \frac{d^2}{\sin^2(\beta)} \right) + \alpha \cdot 2 \cdot \tau_i \cdot d \cdot L}{L \left( L - \frac{d}{\tan(\beta)} \right)} > (\sigma_v + d \cdot \gamma) \tag{4} \]

or:

\[ (\sigma_v + d \cdot \gamma) > \frac{\frac{\sigma_i \cdot d^2 + \alpha \cdot 2 \cdot \tau_i \cdot d \cdot L \cdot \sin^2(\beta)}{L \cdot \left( L - d \cdot \cot(\beta) \right) \cdot \sin^2(\beta)}} \tag{5} \]
APPENDIX C

Interface Elements

An interface or slip-plane model is available to represent distinct interfaces between two or more portions of the grid. The interfaces are planes upon which slip and/or separation are allowed, thereby simulating the presence of faults, joints or frictional boundaries.

An interface is represented as a normal and shear stiffness between two planes which may contact one another (figure A).

![Interface Diagram]

Figure A: An interface represented by sides A and B, connected by shear ($k_s$) and normal ($k_n$) stiffness springs

Where:
- $S$ = slider;
- $T$ = tensile strength;
- $k_n$ = normal stiffness; and
- $k_s$ = shear stiffness.

Three options are available for specifying the conditions of the interface, as follows:

**Glued Interfaces** – if interfaces are declared glued, no slip or opening is allowed, but elastic displacement still occurs, according to the given stiffnesses.

**Coulomb Shear-Strength** – the Coulomb shear-strength criterion limits the shear force by the following relation:
Appendix C – Interface Elements

\[ F_{s \text{ max}} = c \cdot L + F_{n} \cdot \tan(\phi) \]

Where:
- \( c \) = cohesion (in stress units) along the interface,
- \( L \) = effective contact length, and
- \( \phi \) = friction angle of interface surfaces. In addition, the interface may dilate at the onset of slip (non-elastic sliding). Dilation is governed in the Coulomb model by a specified dilation angle, \( \psi \).

**Tension Bond** – If a (positive) tensile bond strength is specified for an interface, each segment of the interface acts as if it is glued (elastic response only) while the magnitude of the tensile normal stress is below the bond strength. If the magnitude of the tensile normal stress of a segment exceeds the bond strength, the bond breaks for that segment, and the segment behaves thereafter as unbonded (separation and slip allowed, in the normal way).

The apparent stiffness (expressed in stress-per-distance units) of a zone in the normal direction is:

\[
\text{max} \left( \frac{K + \frac{4}{3} \cdot G}{\Delta z_{\text{min}}} \right)
\]

Where:
- \( K \) = bulk modulus,
- \( G \) = shear modulus, and
- \( \Delta z_{\text{min}} \) = is the smallest width of an adjoining zone in the normal direction.

Bulk modulus, \( K \), and shear modulus, \( G \), are related to Young's modulus \( E \), and Poisson's ratio \( \nu \), by:
Appendix C – Interface Elements

\[ K = \frac{E}{3 \cdot (1 - 2 \cdot \nu)} \quad 3 \]

\[ G = \frac{E}{2 \cdot (1 + \nu)} \quad 4 \]