ROCK MASS CHARACTERIZATION

and Its Application to Assessment of Unsupported Underground Openings.

Thesis submitted by

NORBERT RICHARD PRZEMYSLAW BACZYNSKI
B.Sc.(Hons)(ANU), M.Sc.(JCU), Dip. Datamet.(UNISA)

November 1980

as partial fulfilment of the requirements for
the Degree of Doctor of Philosophy in
the Department of Mining & Metallurgy at the
University of Melbourne
The research study centres on the stability assessment of unsupported, sub-level open stopes located within the silver-lead-zinc orebodies at the Mount Isa Mine in Queensland, Australia.

A systematic approach is adopted for characterization of rock masses and a statistically-based model is formulated to account for local variability in geological structure. Investigations suggest that the degree of fracture clustering observed in some masses may not have resulted from a random process. A zonal model is proposed for spatial distribution of these weakness planes. The general validity of the assumed model with respect to the dolomitic shales at the mine is confirmed by computer simulation of fracture distributions within three-dimensional blocks. The computer program used in the simulation process has been developed as part of the project.

Mechanical properties of the mass are derived on the basis of the statistical model for geological structure and various published rock mass classification systems. A computer program is used to expedite these analyses. Alternate approaches for derivation of properties are mentioned.

Several techniques are employed to assess the significance of statistically variable material properties on stability of the stopes. These include rock mass classifications, two-dimensional finite elements with elasto-plastic behaviour and sequential excavation, and the probability of failure analysis. Limitations of each technique are discussed.
Some consideration is given to broadly reviewing past mining experience within stopes at the mine.

Results of analyses demonstrate that a greater depth of understanding and appreciation of rock mass behaviour may be gained from models with locally variable material properties. A reasonable degree of correlation is achieved between model and prototype response.
DECLARATION

I declare that this thesis is my own work. It has not been submitted in any form for another degree or diploma at any other university or institute of tertiary education.

Information derived from the published or unpublished work of others has been acknowledged in the text and a list of references is given.

Norbert R.P. Baczyński

25th November 1980.
ACKNOWLEDGEMENTS

Dr. G.D. Base, Mr. W.E. Bamford, Dr. J.R. Barrett and Mr. D.R. Willoughby - staff members of the University of Melbourne and the CSIRO Division of Applied Geomechanics - were involved or were nominated at various times to act in a supervisory and coordinating capacity.

Special thanks are extended to Dr. J.R. Barrett and Mr. W.E. Bamford for their active guidance of the research project.

Thanks are due to Mr. M.C. Bridges of Mount Isa Mines Limited for the interest in statistical approaches to geomechanics that he managed to spark in the writer during the early 1970's.

The writer thanks Mr. F. Leahy of Mount Isa Mines Limited for his continued interest and support of the work. Mr. R. Cowling of the same company, Dr. G. Beer of the University of Queensland and Mr. B. Perkins of the CSIRO Division of Applied Geomechanics offered their enthusiastic assistance at various times during the course of the project.

The work was basically funded from a research grant awarded by Mount Isa Mines Limited. The cost of all travel, field accommodation and on-site computing was borne by the Company. The assistance offered by computing and geological staff at the mine is gratefully acknowledged. Without the Company's generous and kind assistance this project would not have been possible.

The grant was administered by the CSIRO, who also freely provided their office, library and computer facilities. This arrangement ensured a most congenial working environment during the term of the studies.
Funding of the work was supplemented via a single award of the Henry DeWitt Smith Scholarship (American Institute of Mining, Metallurgical and Petroleum Engineers) and a partial award of the Gilbert Rigg Scholarship (University of Melbourne). The former was initially granted in 1975 to pursue the postgraduate programme overseas. However, it was later extended to permit studies at the University of Melbourne during 1977 - 1980. The writer thanks Mr. R.J. Searls of Newmont Proprietary Limited for his interest in the project and for his patience during finalization of study arrangements. The award helped to ease the cost of establishment in Melbourne.

I thank my wife Carol for her support during the years that she has had to live with this project.
## INDEX

<table>
<thead>
<tr>
<th>Paragraph</th>
<th>Subject</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>Definition of Problem and Project Objectives</td>
<td>1</td>
</tr>
<tr>
<td>1.2</td>
<td>Description of Field Site</td>
<td>2</td>
</tr>
<tr>
<td>1.3</td>
<td>Conceptual Basis Adopted for Model</td>
<td>6</td>
</tr>
<tr>
<td>1.3.1</td>
<td>General Implications Inherent in Modelling</td>
<td>6</td>
</tr>
<tr>
<td>1.3.2</td>
<td>Characterization of Mine Shales</td>
<td>8</td>
</tr>
<tr>
<td>1.4</td>
<td>Structure of Thesis</td>
<td>10</td>
</tr>
<tr>
<td>1.5</td>
<td>Constraints on Complexity of Assumed Model</td>
<td>11</td>
</tr>
<tr>
<td>1.5.1</td>
<td>Areas Investigated</td>
<td>11</td>
</tr>
<tr>
<td>1.5.2</td>
<td>Derivation of Rock Mass Properties</td>
<td>12</td>
</tr>
<tr>
<td>1.5.3</td>
<td>Stability Analyses</td>
<td>13</td>
</tr>
</tbody>
</table>

## CHAPTER 1 - INTRODUCTION

### 2.1 Strength of the Intact Rock

- **2.1.1** Unconfined Compressive Strength 18
- **2.1.2** Tensile Strength 23
- **2.1.3** Shear Strength, \( S_0 \), and Angle of Internal Friction \( \phi_i \) 23

### 2.2 Modulus of Elasticity

### 2.3 Poisson's Ratio

### 2.4 Strain at Failure

### 2.5 Summary

### 2.6 Summary

### 2.7 Conclusion

### 2.8 Acknowledgements

### 2.9 References

### 2.10 Appendices

### A1 Model for Intact Rock Component of Rock Mass

### A2 Characterization of Mine Shales

### A3 General Implications Inherent in Modelling

### A4 Constraints on Complexity of Assumed Model

### A5 Areas Investigated

### A6 Derivation of Rock Mass Properties

### A7 Stability Analyses

### A8 Summary

### A9 Conclusion

### A10 Acknowledgements

### A11 References

### A12 Appendices
### INDEX

<table>
<thead>
<tr>
<th>Paragraph</th>
<th>Subject</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CHAPTER 3 - PHYSICAL CHARACTERISTICS OF GEOLOGICAL DISCONTINUITIES</strong></td>
<td>28</td>
<td></td>
</tr>
<tr>
<td>3.1</td>
<td>Source of Data</td>
<td>28</td>
</tr>
<tr>
<td>3.2</td>
<td>Scope of Model</td>
<td>29</td>
</tr>
<tr>
<td>3.3</td>
<td>Characteristics of Discontinuities</td>
<td>31</td>
</tr>
<tr>
<td>3.3.1</td>
<td>Orientation</td>
<td>31</td>
</tr>
<tr>
<td>3.3.2</td>
<td>Spacing</td>
<td>34</td>
</tr>
<tr>
<td>3.3.3</td>
<td>Shape and Continuity</td>
<td>38</td>
</tr>
<tr>
<td>3.3.4</td>
<td>Frictional Properties of Fractures</td>
<td>42</td>
</tr>
<tr>
<td><strong>CHAPTER 4 - MODEL FOR SPATIAL DISTRIBUTION OF FRACTURES</strong></td>
<td>46</td>
<td></td>
</tr>
<tr>
<td>4.1</td>
<td>General Comments</td>
<td>47</td>
</tr>
<tr>
<td>4.2</td>
<td>&quot;Even&quot; and &quot;Variable&quot; Models</td>
<td>47</td>
</tr>
<tr>
<td>4.3</td>
<td>&quot;Random&quot; Model</td>
<td>48</td>
</tr>
<tr>
<td>4.4</td>
<td>Testing of &quot;Random&quot; Model</td>
<td>50</td>
</tr>
<tr>
<td>4.5</td>
<td>Basis of &quot;Zonal&quot; Model</td>
<td>53</td>
</tr>
<tr>
<td>4.6</td>
<td>Derivation of &quot;Zonal&quot; Model Parameters</td>
<td>54</td>
</tr>
<tr>
<td>4.6.1</td>
<td>Field Investigations</td>
<td>54</td>
</tr>
<tr>
<td>4.6.2</td>
<td>Evaluation of Data</td>
<td>55</td>
</tr>
<tr>
<td>4.7</td>
<td>Some Statistical Considerations</td>
<td>58</td>
</tr>
<tr>
<td>4.8</td>
<td>Spatial Relationship Observed at the Mine</td>
<td>62</td>
</tr>
<tr>
<td>4.9</td>
<td>Model Testing</td>
<td>69</td>
</tr>
<tr>
<td>4.10</td>
<td>Spatial Model for Bedding Plane Partings</td>
<td>77</td>
</tr>
</tbody>
</table>
## INDEX

<table>
<thead>
<tr>
<th>Paragraph</th>
<th>Subject</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>CHAPTER 5</td>
<td>MODEL FOR STRUCTURAL VARIABILITY OF LARGE BLOCKS</td>
<td>81</td>
</tr>
<tr>
<td>5.1</td>
<td>Model for Each Fracture Set</td>
<td>81</td>
</tr>
<tr>
<td>5.2</td>
<td>Combined Model for Rock Mass</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CHAPTER 6</td>
<td>ROCK MASS PROPERTIES</td>
<td>96</td>
</tr>
<tr>
<td>6.1</td>
<td>Method of Analysis</td>
<td>96</td>
</tr>
<tr>
<td>6.2</td>
<td>Computer Simulation</td>
<td>97</td>
</tr>
<tr>
<td>6.3</td>
<td>Model for RQD-Index</td>
<td>98</td>
</tr>
<tr>
<td>6.4</td>
<td>Basis of Model for Rock Mass Strength</td>
<td>100</td>
</tr>
<tr>
<td>6.4.1</td>
<td>Intact Rock Mass</td>
<td>100</td>
</tr>
<tr>
<td>6.4.2</td>
<td>Discontinuous Rock Mass</td>
<td>101</td>
</tr>
<tr>
<td>6.5</td>
<td>Basis of Model for Rock Mass Modulus</td>
<td>103</td>
</tr>
<tr>
<td>6.5.1</td>
<td>Modulus of Intact Rock</td>
<td>104</td>
</tr>
<tr>
<td>6.5.2</td>
<td>Theoretical Approach to Rock Mass Modulus</td>
<td>105</td>
</tr>
<tr>
<td>6.5.3</td>
<td>Rock Mass Modulus</td>
<td>106</td>
</tr>
<tr>
<td>6.6</td>
<td>Poisson's Ratio Assumed for Rock Mass</td>
<td>108</td>
</tr>
<tr>
<td>6.7</td>
<td>Statistical Model for Material Properties</td>
<td>109</td>
</tr>
<tr>
<td>6.7.1</td>
<td>Definition of Domains</td>
<td>109</td>
</tr>
<tr>
<td>6.7.2</td>
<td>Material Types</td>
<td>110</td>
</tr>
<tr>
<td>6.7.3</td>
<td>Frequency of Occurrence</td>
<td>110</td>
</tr>
</tbody>
</table>
### INDEX

<table>
<thead>
<tr>
<th>Paragraph</th>
<th>Subject</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CHAPTER 7 - STABILITY ANALYSIS</strong></td>
<td></td>
<td>115</td>
</tr>
<tr>
<td>7.1</td>
<td>Rock Mass Classifications</td>
<td>116</td>
</tr>
<tr>
<td>7.1.1</td>
<td>Classifications Considered</td>
<td>116</td>
</tr>
<tr>
<td>7.1.2</td>
<td>Estimates of Stable Hanging-wall Spans</td>
<td>117</td>
</tr>
<tr>
<td>7.2</td>
<td>Finite Element Modelling</td>
<td>125</td>
</tr>
<tr>
<td>7.2.1</td>
<td>Choice of Computer Program</td>
<td>125</td>
</tr>
<tr>
<td>7.2.2</td>
<td>Complementary Programs</td>
<td>127</td>
</tr>
<tr>
<td>7.2.3</td>
<td>Geometry of Model</td>
<td>127</td>
</tr>
<tr>
<td>7.2.4</td>
<td>Mesh Formulation</td>
<td>129</td>
</tr>
<tr>
<td>7.2.5</td>
<td>Details of Base Mesh</td>
<td>130</td>
</tr>
<tr>
<td>7.2.6</td>
<td>Mesh Types</td>
<td>133</td>
</tr>
<tr>
<td>7.2.7</td>
<td>Accuracy of Mesh</td>
<td>135</td>
</tr>
<tr>
<td>7.2.8</td>
<td>Stress Field Assumed in Analysis</td>
<td>137</td>
</tr>
<tr>
<td>7.2.9</td>
<td>Extent of Finite Element Modelling</td>
<td>137</td>
</tr>
<tr>
<td></td>
<td>Programme</td>
<td>137</td>
</tr>
<tr>
<td>7.3</td>
<td>Results of Finite Element Modelling</td>
<td>140</td>
</tr>
<tr>
<td>7.3.1</td>
<td>Extent of Plastic Yield</td>
<td>140</td>
</tr>
<tr>
<td>7.3.2</td>
<td>Displacements</td>
<td>153</td>
</tr>
<tr>
<td>7.3.3</td>
<td>Stresses</td>
<td>157</td>
</tr>
<tr>
<td>7.4</td>
<td>Probability of Failure Analysis</td>
<td>164</td>
</tr>
<tr>
<td>7.4.1</td>
<td>Description of Technique</td>
<td>164</td>
</tr>
<tr>
<td>7.4.2</td>
<td>Assessment of Probability</td>
<td>166</td>
</tr>
<tr>
<td>7.4.3</td>
<td>Advantages and Limitations of Method</td>
<td>183</td>
</tr>
</tbody>
</table>
## CHAPTER 8 - COMPARISON BETWEEN MODEL AND PROTOTYPE RESPONSE

<table>
<thead>
<tr>
<th>Paragraph</th>
<th>Subject</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.1</td>
<td>Practical Difficulties</td>
<td>186</td>
</tr>
<tr>
<td>8.2</td>
<td>Scope of Analysis</td>
<td>188</td>
</tr>
<tr>
<td>8.3</td>
<td>Observed Response</td>
<td>189</td>
</tr>
<tr>
<td>8.4</td>
<td>Rock Mass Classification Predictions</td>
<td>194</td>
</tr>
<tr>
<td>8.5</td>
<td>Finite Element Predictions</td>
<td>196</td>
</tr>
<tr>
<td>8.6</td>
<td>Probability of Failure Analysis Predictions</td>
<td>198</td>
</tr>
</tbody>
</table>

## CHAPTER 9 - SUMMARY AND DISCUSSION

<table>
<thead>
<tr>
<th>Paragraph</th>
<th>Subject</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.1</td>
<td>Background Considerations</td>
<td>199</td>
</tr>
<tr>
<td>9.2</td>
<td>Basis of Adopted Model</td>
<td>201</td>
</tr>
<tr>
<td>9.3</td>
<td>Derivation of Model Parameters</td>
<td>201</td>
</tr>
<tr>
<td>9.4</td>
<td>Stability Analysis</td>
<td>203</td>
</tr>
<tr>
<td>9.5</td>
<td>Conclusions</td>
<td>205</td>
</tr>
<tr>
<td>9.6</td>
<td>Recommendations</td>
<td>208</td>
</tr>
</tbody>
</table>

## REFERENCES

<table>
<thead>
<tr>
<th>Additional References</th>
</tr>
</thead>
</table>

## APPENDIX I - ROCK MASS CLASSIFICATION INPUT PARAMETERS A1-A5

<table>
<thead>
<tr>
<th>Appendix</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1.1</td>
<td>Bieniawski's RMR Method</td>
<td>A1</td>
</tr>
<tr>
<td>A1.2</td>
<td>Laubscher and Taylor's Modified RMR Method</td>
<td>A2</td>
</tr>
<tr>
<td>A1.3</td>
<td>Barton's Q-Method</td>
<td>A4</td>
</tr>
</tbody>
</table>
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table No.</th>
<th>Description</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Characteristics of Fracture Sets.</td>
<td>33</td>
</tr>
<tr>
<td>2</td>
<td>Mean Intensity of Fractures per Unit Volume.</td>
<td>72</td>
</tr>
<tr>
<td>3</td>
<td>Relationship Between C-Factor and Intensity of Fracturing per Metre.</td>
<td>103</td>
</tr>
<tr>
<td>4</td>
<td>Rock Mass Material Properties.</td>
<td>111</td>
</tr>
<tr>
<td>5</td>
<td>Average Material Types Used in Modelling.</td>
<td>113</td>
</tr>
<tr>
<td>6</td>
<td>Hanging-wall Spans (Metres) on Basis of Rock Mass Classifications.</td>
<td>118</td>
</tr>
<tr>
<td>7</td>
<td>Magnitude of Pre-Mining Stress Field Assumed in Modelling.</td>
<td>138</td>
</tr>
<tr>
<td>8</td>
<td>Sensitivity Testing: Different Initial Stresses.</td>
<td>138</td>
</tr>
<tr>
<td>9</td>
<td>Finite Element Modelling Programme Using VISPLAS.</td>
<td>139</td>
</tr>
<tr>
<td>10</td>
<td>Stability of 5 Orebody Stope with Uniform (Homogeneous) Material Properties.</td>
<td>142</td>
</tr>
<tr>
<td>11</td>
<td>Stability of 5 Orebody Stope with Heterogeneous (Statistically Variable) Material Properties.</td>
<td>143</td>
</tr>
</tbody>
</table>
### LIST OF TABLES

<table>
<thead>
<tr>
<th>Table No.</th>
<th>Description</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>Stability of 11 Orebody Stope with Uniform (Homogeneous) Material Properties.</td>
<td>144</td>
</tr>
<tr>
<td>13</td>
<td>Stability of 11 Orebody Stope with Heterogeneous (Statistically Variable) Material Properties.</td>
<td>145</td>
</tr>
<tr>
<td>14</td>
<td>Case Study: Variations in the Width of Stopes (in Metres).</td>
<td>193</td>
</tr>
<tr>
<td>15</td>
<td>Case Study: Stability of Hanging-walls &amp; Abutments.</td>
<td>193</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure No.</th>
<th>Description</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Typical Cross-Section Through Northern Mining Area.</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>Diagrammatic Layout of a Sublevel Open Stope in Silver-Lead-Zinc Orebodies.</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td>Formulation of a Geotechnical Model.</td>
<td>7</td>
</tr>
<tr>
<td>4</td>
<td>Conceptual Basis for Rock Mass Characterization.</td>
<td>9</td>
</tr>
<tr>
<td>5</td>
<td>Relationship Between Core-Bedding Angle and the Unconfined Compressive Strength of Cores.</td>
<td>20</td>
</tr>
<tr>
<td>6</td>
<td>Strength of 61 Specimens of Mineralized Shale with Angle of 31-45° Between Loading Axis and Bedding Plane.</td>
<td>22</td>
</tr>
<tr>
<td>7</td>
<td>Principal Fracture Sets.</td>
<td>32</td>
</tr>
<tr>
<td>8</td>
<td>Cumulative Frequency Plot on Logarithmic Probability Paper for the Spacing of Fracture Set 2 and 3 (Diamond Drill Core Data).</td>
<td>35</td>
</tr>
<tr>
<td>9</td>
<td>Relationship Between Median Frequency of Extension Fractures and Frequency of Bedding Plane Partings.</td>
<td>37</td>
</tr>
<tr>
<td>10</td>
<td>Cumulative Frequency Plot on Logarithmic Probability Paper for Continuity of Fracture Set No. 2 in Directions Parallel and Normal to Bedding of Mine Shales (Line Sample Data).</td>
<td>40</td>
</tr>
</tbody>
</table>
### LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure No.</th>
<th>Description</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>Cumulative Frequency Plot on Logarithmic Probability Paper for Continuity of Bedding Plane Partings (Line Sample Data).</td>
<td>41</td>
</tr>
<tr>
<td>12</td>
<td>Cumulative Frequency Distribution on Normal Probability Paper for Proportion of Graphitic Bedding Plane Partings in Samples of 10 Partings (Total Sample Size = 1000).</td>
<td>44</td>
</tr>
<tr>
<td>13</td>
<td>Pattern of Fracture Traces Generated on Basis of &quot;Random&quot; Model for Spatial Distribution of Fracture Plane Centres.</td>
<td>51</td>
</tr>
<tr>
<td>14</td>
<td>Cumulative Frequency Distribution on Logarithmic Probability Paper for Spacing of Adjacent Fractures Derived on Basis of &quot;Random&quot; Spatial Model.</td>
<td>52</td>
</tr>
<tr>
<td>15</td>
<td>Unit Areas with respect to a Particular Set.</td>
<td>56</td>
</tr>
<tr>
<td>16</td>
<td>Conceptual Relationship: Influence of Immediately Adjacent Areas on Observed Fracture Intensity Within a Designated Unit Area.</td>
<td>57</td>
</tr>
<tr>
<td>17</td>
<td>Joint and Marginal Probability Density Functions of Continuous Random Variables X and Y.</td>
<td>61</td>
</tr>
</tbody>
</table>
### LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure No.</th>
<th>Description</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>18</td>
<td>Cumulative Frequency Plot on Logarithmic Probability Paper for Local Variability in the Intensity of Fracture Set No. 1.</td>
<td>64</td>
</tr>
<tr>
<td>19</td>
<td>Probability Density Functions for Intensities of Fractures Observed Adjacent to Unit Areas with a Known Intensity &quot;i&quot; in Direction Parallel to Average Orientation of Set No. 1.</td>
<td>65</td>
</tr>
<tr>
<td>20</td>
<td>Probability Density Functions for Intensities of Fractures Observed Adjacent to Unit Areas with Known Intensity &quot;i&quot; in Direction Normal to Average Orientation of Set No. 1.</td>
<td>66</td>
</tr>
<tr>
<td>21</td>
<td>Conceptual Relationship Between Test-Block and Master-Block in Computer Program CRACKS-2.</td>
<td>70</td>
</tr>
<tr>
<td>22</td>
<td>Typical Fracture Pattern Generated for Fracture Set No. 1 on Basis of &quot;Zonal&quot; Model.</td>
<td>73</td>
</tr>
<tr>
<td>23</td>
<td>Cumulative Frequency Distribution on Logarithmic Probability Paper for Spacing Between Adjacent Fractures of Set No. 1 Derived on Basis of &quot;Zonal&quot; Spatial Model.</td>
<td>74</td>
</tr>
<tr>
<td>Figure No.</td>
<td>Description</td>
<td>Page No.</td>
</tr>
<tr>
<td>-----------</td>
<td>------------------------------------------------------------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>24</td>
<td>Fracture Pattern for Set Nos.1-6 Generated on Several Planes through a 10m x 8m x 6m Test Block on Basis of &quot;Zonal&quot; Spatial Model.</td>
<td>76</td>
</tr>
<tr>
<td>25</td>
<td>Cumulative Frequency Plot on Normal Probability Paper for Local Variability in the Intensity (per metre) of Fracture Set No.11 (Bedding Plane Partings) in the Immediate Hanging-wall of Orebody No.11.</td>
<td>79</td>
</tr>
<tr>
<td>26</td>
<td>Cumulative Frequency Plot on Normal Probability Paper for Variability in the Mean Intensity (per metre) of Fracture Set No.11 (Bedding Plane Partings) Between Stope Hanging-walls in Orebody No.11.</td>
<td>80</td>
</tr>
<tr>
<td>27</td>
<td>Approximate Relationship Between Volume of Rock Mass &amp; the Mean and Standard Deviation of Normal Probability Density Functions for Intensity of Fracture Set Nos.1-11 ($m^2/m^3$).</td>
<td>83-87</td>
</tr>
<tr>
<td>28</td>
<td>Cumulative Frequency Plot on Logarithmic Probability Paper for Local Variability in the Intensity of Fracture Set Nos.1-10 Combined.</td>
<td>91</td>
</tr>
<tr>
<td>29</td>
<td>Cumulative Frequency Plot on Probability Paper for Local Variability in the Intensity of Fracture Set Nos.1-11 Combined within the Hanging-wall of Orebody No.5.</td>
<td>92</td>
</tr>
<tr>
<td>Figure No.</td>
<td>Description</td>
<td>Page No.</td>
</tr>
<tr>
<td>------------</td>
<td>-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>30</td>
<td>Cumulative Frequency Plot on Normal Probability Paper for Local Variability in the Intensity of Fracture Set Nos.1-11 Combined within the Hanging-wall of Orebody No.11</td>
<td>93</td>
</tr>
<tr>
<td>31</td>
<td>Approximate Relationship Between Volume of Rock Mass &amp; the Mean and Standard Deviation of Normal Probability Density Functions for the Intensity of Fracture Set Nos.1-10 and 1-11 Combined within the Hanging-walls of Orebody Nos. 5, 7 and 11, respectively (m²/m³).</td>
<td>94-95</td>
</tr>
<tr>
<td>32</td>
<td>Relationship Between Fracture Frequency and RQD-Index: Mount Isa Mine.</td>
<td>99</td>
</tr>
<tr>
<td>32A</td>
<td>Derivation of Rock Mass Cohesion.</td>
<td>102c</td>
</tr>
<tr>
<td>33</td>
<td>Cumulative Frequency Distributions Plotted on Probability Paper for the Occurrence of Material Types within Unit Volumes (1.0 m³) of the Rock Mass.</td>
<td>114</td>
</tr>
<tr>
<td>Figure No.</td>
<td>Description</td>
<td>Page No.</td>
</tr>
<tr>
<td>-----------</td>
<td>---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>---------</td>
</tr>
<tr>
<td>40</td>
<td>Cross-Section of Underground Sub-level Open Stope Represented by Finite Element Model.</td>
<td>128</td>
</tr>
<tr>
<td>41</td>
<td>Base Mesh.</td>
<td>131-132</td>
</tr>
<tr>
<td>42</td>
<td>Dimensions of Various Mesh-Types Used in Analysis.</td>
<td>134</td>
</tr>
<tr>
<td>43</td>
<td>Mid-span Displacement of Hanging-wall at Various Excavation Stages: Comparison between Boundary Integral and Finite Element Programs.</td>
<td>136</td>
</tr>
<tr>
<td>Figure No.</td>
<td>Description</td>
<td>Page No.</td>
</tr>
<tr>
<td>-----------</td>
<td>-------------------------------------------------------------------------------------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>44</td>
<td>Key Diagram Indicating Elements that are Considered to be Located within Hanging-wall(x) and Abutment(@) Domains.</td>
<td>141</td>
</tr>
<tr>
<td>45</td>
<td>Orebody No.5: Cumulative Frequency Distributions (plotted on Probability Paper) for Number of Yielding Elements in Hanging-wall of Stope with Uniform and Heterogeneous Material Properties.</td>
<td>146</td>
</tr>
<tr>
<td>46</td>
<td>Orebody No.11: Cumulative Frequency Distributions (plotted on Probability Paper) for Number of Yielding Elements in Hanging-wall of Stope with Uniform and Heterogeneous Material Properties.</td>
<td>147</td>
</tr>
<tr>
<td>47</td>
<td>Orebody No.5:Extent of Plastic Yielding Around Stope with Statistically Variable Material Properties.</td>
<td>148</td>
</tr>
<tr>
<td>48</td>
<td>Orebody No.11: Extent of Plastic Yielding Around Stope with Statistically Variable Material Properties.</td>
<td>149</td>
</tr>
<tr>
<td>49</td>
<td>Extent of Plastic Yield in Model with Mean Uniform (Homogeneous) Material Properties: Orebody No.11.</td>
<td>151</td>
</tr>
<tr>
<td>50</td>
<td>Typical Example of the Extent of Plastic Yield in Model with Statistically Heterogeneous Material Properties: Orebody No.11.</td>
<td>152</td>
</tr>
<tr>
<td>51</td>
<td>Orebody No.5: Mid-span Hanging-wall Displacements.</td>
<td>154</td>
</tr>
<tr>
<td>Figure No.</td>
<td>Description</td>
<td>Page No.</td>
</tr>
<tr>
<td>-----------</td>
<td>------------------------------------------------------------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>52</td>
<td>Orebody No.11: Mid-span Hanging-wall Displacements.</td>
<td>155</td>
</tr>
<tr>
<td>53</td>
<td>Orebody No.5: Mean Material Properties: Mid-abutment Displacements at Various Depths.</td>
<td>156</td>
</tr>
<tr>
<td>54</td>
<td>Orebody No.5: Model with Uniform Material Properties: Principal Stress and Displacement Vectors.</td>
<td>158-159</td>
</tr>
<tr>
<td>55</td>
<td>Orebody No.11: Model with Uniform Material Properties: Plot of Principal Stress and Displacement Vectors.</td>
<td>160-161</td>
</tr>
<tr>
<td>56</td>
<td>Example of Typical Stresses Observed in Finite Element Models with Uniform and Statistically Heterogeneous Material Properties.</td>
<td>163</td>
</tr>
<tr>
<td>57</td>
<td>A Hypothetical Example of Probability of Failure Analysis.</td>
<td>167</td>
</tr>
<tr>
<td>58</td>
<td>Orebody No.5: Contours of Probability for Occurrence of Local Ground Failure: Y-stress = 36 MPa, X-stress = 16 MPa.</td>
<td>168-172</td>
</tr>
<tr>
<td>59</td>
<td>Orebody No.11: Contours of Probability for Occurrence of Local Ground Failure: Y-stress = 36 MPa, X-stress = 16 MPa.</td>
<td>173-177</td>
</tr>
<tr>
<td>60</td>
<td>Orebody No.5: Excavation Stage No.5: Contours of Probability for Occurrence of Local Ground Failure: Y-stress = 26 MPa, X-stress = 16 MPa.</td>
<td>178</td>
</tr>
<tr>
<td>Figure No.</td>
<td>Description</td>
<td>Page No.</td>
</tr>
<tr>
<td>-----------</td>
<td>------------------------------------------------------------------------------------------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>61</td>
<td>Orebody No.5: Excavation Stage No.5: Contours of Probability for Occurrence of Local Ground Failure: Y-stress = 16 MPa, X-stress = 16 MPa.</td>
<td>179</td>
</tr>
<tr>
<td>62</td>
<td>Orebody No.5: Excavation Stage No.5: Contours of Probability for Occurrence of Local Ground Failure: Y-stress = 12 MPa, X-stress = 16 MPa.</td>
<td>180</td>
</tr>
<tr>
<td>63</td>
<td>Orebody No.5: Excavation Stage No.5: Contours of Probability for Occurrence of Local Ground Failure: Y-stress = 12 MPa, X-stress = 26 MPa.</td>
<td>181</td>
</tr>
<tr>
<td>64</td>
<td>Schematic Representation of Probability of Failure Analysis for Extensive Zones.</td>
<td>182</td>
</tr>
<tr>
<td>65</td>
<td>Orebody Nos. 5, 7 and 11: Relationship Between Strike Span of Hanging-walls and Width of Stopes.</td>
<td>190</td>
</tr>
<tr>
<td>66</td>
<td>Orebody Nos. 5 and 7 Combined: Cumulative Frequency Distribution plotted on Probability Paper for the Strike Span of Hanging-walls.</td>
<td>191</td>
</tr>
</tbody>
</table>
CHAPTER 1 - INTRODUCTION

1.1 Definition of Problem and Project Objectives

Most rock masses display several common attributes:

(1) They are physical systems whose principal components are intact rock and geological discontinuities.

(2) They are rarely, if ever, homogeneous (uniform) throughout. Variability is apparent at all scales and levels of detail.

(3) Their response under load may be potentially complex.

Logically, the above attributes tend to militate against the characterization of real rock masses in terms of simple, homogeneous systems with single sets of material properties.

Accordingly, the objectives of the present study are to examine the geological variability within a typical rock mass and then to assess the importance of this variability on the stability of openings located within such heterogeneous systems. The work is based on field investigations undertaken in the silver-lead-zinc orebodies at the Mount Isa Mine in north-western Queensland, Australia.

Three basic phases are involved in the study:

(1) Formulation of a statistically-based model to adequately account for the observed structural variability within the mine shales.
(2) Equation of the structural model with a corresponding model for variability in mechanical properties of the rock mass.

(3) Examination of the effect and the significance, if any, of a heterogeneous model in stability analyses; with special reference to the hangingwalls of unsupported, sublevel open stopes in the silver-lead-zinc orebodies at the mine.

1.2 Description of Field Site

The sedimentary rocks which contain the Mount Isa deposits are part of the Lower Proterozoic strata of the Australian Precambrian Shield. The economic mineralization is confined to the Urquhart Shale Formation within the Mount Isa Group.

Structurally, the Group is located on the western limb of a north plunging anticline, the axis of which trends towards north-south about 19 kilometres to the east of Mount Isa. A predominant north-south strike and a 60-65° westerly dip persists for some 28 kilometres northwards and 32 kilometres southwards from the mine.

On the local scale, the Urquhart Shales are composed of bedded, pale and dark grey, fine grained dolomitic and volcanic shales, minor tuffaceous bands, and silica-rich breccias.

The silver-lead-zinc orebodies are restricted to unaltered shale areas. The copper orebodies occur in zones of recrystallized shale and deformed Urquhart Shale locally known as silica-dolomite.
The silver-lead-zinc orebodies occur in discontinuous, concordant bands which may extend over a strike length of 1000 metres and over a down-dip distance of 600 metres. Width of individual orebodies ranges between 1.5 and 45 metres. Concentrations of economic mineralization are known in 15 distinct groups of stratigraphic horizons. Spatially, the orebodies are arranged in an en-echelon pattern. A typical vertical cross-section normal to stratigraphy and looking northwards is presented in Figure 1.

Minor folding is common in the mine, especially in mineralized areas. Major folding is confined to distinct zones. Strike faulting and shearing, transverse faulting and jointing are locally well developed in the mine shales.

Various mining techniques have been used at the mine over the last fifty years. They include glory-hole mining, sublevel open stoping, a combination of sublevel and shrink stoping, and a mechanised cut-and-fill method (locally known as MICAF). The sublevel open stoping and cut-and-fill methods are currently being employed to extract the silver-lead-zinc orebodies.

Individual sublevel open stopes may extend over a strike length of 80 metres, over a down-dip distance of 250 metres and over a width (in the direction normal to bedding) of 45 metres. The dimensions of the average stope are much smaller, with the corresponding distances of about 30 metres, 150 metres and 20 metres, respectively. Commonly, the walls and crown of sublevel open stopes are not supported or reinforced during mining. However, fill is now generally placed in the stopes at the end of ore extraction.

The geometry and layout of a typical open stope is illustrated in Figure 2.
FIGURE 1. - Typical Cross-Section Through Northern Mining Area.
FIGURE 2. - Diagrammatic Layout of a Sub-Level Open Stope in Silver-Lead-Zinc Orebodies
1.3 Conceptual Basis Adopted for Model

1.3.1 General Implications Inherent in Modelling

Any stability analysis of a rock mass system is a conceptual process. This arises because the analysis invariably involves the idealization of a real rock mass by means of a simplified model.

Taha (1976) has schematically depicted the level of abstraction that is commonly involved in the formulation of a model from a real-life situation. The appropriate diagram is reproduced in Figure 3.

The "assumed real world" is abstracted from the real situation by concentrating on those variables that one considers or presumes to be dominant in controlling the behaviour of the real system. The model, being an abstraction of the "assumed real world", simplifies the relationship among these variables in a form that is amenable to analysis and testing under different sets of external conditions.

There are limitations associated with any model formulation and the extrapolation of model results to in-situ or field situations.

Although the response of the model may be in accord with the "assumed real world", it does not necessarily follow that the behaviour of the simplified model will adequately represent the true response of the "real world". This arises because the "assumed real world" may not be representative of the "real world".
FORMULATION OF A GEOTECHNICAL MODEL
After Taha (1976).

FIGURE 3.
1.3.2 Characterization of Mine Shales

The basis of the concept adopted in the present study for the characterization of rock masses rests on the following idealization:

ANY LARGE BLOCK OF ROCK MASS MAY BE REPRESENTED BY AN EQUIVALENT SYSTEM OF SMALLER SUB-BLOCKS OR MODULES.

The concept is illustrated schematically in Figure 4.

The above concept is similar to the "systems approach" first formulated in the 1930's by von Bertalanffy. This approach has been more recently cited in literature by Couger and Knapp (1974), Ackoff (1974), Davis (1974) and others with respect to the analysis of information systems.

Where a system is too complex to permit an evaluation of its response as a whole, the problem can be simplified by dividing or factoring the system into subsystems. The use of subsystems may also be termed as a modular or building block concept. The process of factoring is continued to a stage where individual subsystems are of a manageable size.

This reduces the problem in the sense that it is now comprised of modules or blocks, and the response of each of the units may now be more readily assessed.

To a large extent, the response of the entire system is the total sum of the responses of the individual subsystems that are located along "critical paths" within the system. This view is supported by von Bertalanffy (1969) when he states that "...... the properties and modes of action of higher levels are not explicable by the summation of the properties and modes of action of their components
FIGURE 4. - Conceptual Basis for Rock Mass Characterization
taken in isolation. If, however, we know the ensemble of the components and the relations existing between them, then the higher levels are derivable from the components......". In terms of rock masses, the "critical paths" are likely to correspond with areas of high stress gradient, or with zones of high compressive or tensile stresses. Because structural variability within a mass is statistically defined, the interfaces or interrelations between the subsystems along the "critical paths" may also be defined in a similar probabilistic manner.

Thus, the systems approach to problems focuses on the system taken as a whole, not their parts taken separately.

The writer is of the opinion that the systems concept offers a sound approach for characterization of rock masses and for assessing rock mechanics problems. It constitutes a framework for visualizing and tackling the difficult task of defining structural and mechanical variability within rock masses and then extending the process to modelling of the response of the entire mass during mining.

1.4 Structure of Thesis

The results of the present study are presented in five main parts:

(1) A review of intact rock properties as derived from laboratory testing.

(2) Summary of field investigations and the formulation of a model for structural variability within the mine shales.

(3) Derivation of an equivalent model for variability in the continuum material properties of the rock mass.
1.5 Constraints on Complexity of Assumed Model

1.5.1 Areas Investigated

A dominant feature of the dolomitic shales at the mine is the relatively high frequency of "minor" geological structures.

Accordingly, one of the prime considerations in this study has been to examine the characteristics and spatial distribution of various fracture sets within uniformly bedded shales and then to devise a method of assessment whereby it would be possible to include their effect in the analysis.

The scope of the investigation is limited to Orebody Nos. 5, 7 and 11. These orebodies are purposely selected because they are considered to embrace the full range of ground conditions that have been experienced in the past within sublevel open stopes at the mine.

The scope of the analysis is also limited to exclude those areas of
the mine where the rock mass is transected by major, gouge-infilled faults.

1.5.2 Derivation of Rock Mass Properties

It is apparent that field testing offers the only reliable method for evaluating various rock mass parameters (i.e., cohesion, effective peak and residual friction angles, unconfined compressive and tensile strengths, elastic modulus or stiffness, and Poisson's ratio).

Several aspects characterize such tests:

1. The dimensions of the test samples are commonly quite large to ensure that they are representative of the mass.

2. Where the rock mass is locally variable, a large number of tests are necessary to permit meaningful trends to be deduced from the results.

3. Testing programmes are costly and very time consuming.

Because of the above characteristics and difficulties in locating suitable test-sites, most field testing programmes tend to be rather limited in scope. Very few results are reported in technical literature.

Similarly, there was little opportunity to extend the scope of the present study to include such tests.

Estimates of rock mass parameters used in the present analysis are derived on the basis of various published rock mass classifications. Details of the procedure employed to determine these values are discussed in Chapter 6 of this thesis.
The parameters are determined for the range of structurally possible situations within subsystems or unit modules of the dolomitic shales. A computer program GEOCLAS was developed to expedite the assessment. The analysis centres on "Monte Carlo" sampling of probability density functions for the intensity of each fracture set as well as other input parameters and the derivation of appropriate ratings for several classification systems. The process is iterated as many times as considered necessary to derive a model for variability between sub-blocks (of any specified dimensions).

There is no doubt that the above assessment can only hope to yield a crude estimate. Thus, various criticisms may be levelled at this approach. Most of these relate to the simplicity of the ensuing material model and the validity of extrapolating relationships established at other sites to the mass under investigation.

In spite of these limitations, the values may be considered as first-order approximations of the in-situ parameters. They suffice to demonstrate the potential extent of variability that may exist locally within the mine shales. Validation of the assumed values can only be achieved by appropriate field testing.

1.5.3 Stability Analyses

The formulation or choice of model(s) should be mainly governed by two considerations:

(1) The model(s) should approximate as closely as possible the real situation.

(2) The model(s) should be as simple as possible.
The accuracy of prediction and therefore, the usefulness of the results obtained on the basis of modelling, depends to a large extent on whether the selected formulation reflects the realities of the in-situ problem.

The degree of difficulty associated with the incorporation of "minor" structural features in geotechnical models is directly proportional to the scale factor between the continuity and intensity of the structures and the physical dimensions of the excavation being modelled. Low continuity and high frequency of structures confound the task of realistically modelling the mass.

Several potential modelling approaches were examined.

Initial considerations were given to physical modelling. An outline of a tentative testing programme was drawn up and is presented in Baczynski (1977b). A survey of relevant technical literature was undertaken (Baczynski, 1980d). The Australian Coal Industry Research Laboratories at Wollongong were visited in late-1977 to review physical modelling techniques and to inspect testing facilities. Full access was also available to the results of modelling work undertaken at the CSIRO Division of Applied Geomechanics in Melbourne.

Several difficulties became apparent during the review. These centre on:

1. Development of representative equivalent materials to simulate intact rock.

2. Quality control to ensure that uniformity of equivalent materials is maintained throughout the testing programme.

3. Adequate means for simulating gravitational loading.

4. The time element involved in the construction of large, three-dimensional models with high frequencies of non-penetrating discontinuities to simulate geological structure.
(5) Ensuring that model construction extending over several months did not superimpose significant variability on the properties of equivalent materials due to local differences in drying and curing time.

(6) Development of a suitable technique for the incorporation of relatively closed (tight) geological structures.

(7) Adequate simulation of the frictional characteristics of joints and bedding plane partings.

The latter two factors are considered to be especially important. This arises because the strength and response of a model is largely a function of:

(1) Frictional properties of the weakness planes, and

(2) Degree of interlock that exists between the constituent sub-blocks of the model.

It is suspected by the writer that physical models with fairly open discontinuities would have resulted in unrealistic local stress concentrations at the extremities of the weakness planes and the response of the system under load may not have reflected the true behaviour of mine shales.

Several numerical techniques for the assessment of continua and discontinua were also examined.

The two most applicable techniques available for numerical modelling of discontinua are:

(1) Finite element method with joint elements to simulate geological discontinuities, as described by Goodman et al. (1968, 1977) and others, and

(2) Blocky model method developed by Cundall (1971).
The two methods permit a more realistic characterization of rock masses. However, an enormous computational effort is involved in the analysis of even relatively simple systems. This places a severe limitation on the size and complexity of the problem that can be successfully coded and analysed.

Three different approaches were finally selected to assess the stability of stopes at the mine. These are:


2. Two-dimensional, Finite Element Method.

3. Probability of Failure Analysis.

Rock mass classifications that are employed include Bieniawski (1973) RMR-index, Laubscher and Taylor (1976) Modified RMR-index, and the Barton et al. (1974) Q-index.

Considerable difficulty was experienced in securing a finite element program that would model elasto-plastic behaviour and permit sequential excavation of openings. All programs that were examined appeared to perform well in modelling of a single-stage opening in an elastic continuum. However, problems arose in most of them when the analysis was extended to multi-stage models. Errors were mainly of the type that is discussed in Christian and Wong (1973). They relate to the dependency between observed displacements and excavation sequence. Some simple, but potentially significant, logic errors were also detected.

The finite element analysis was undertaken using VISPLAS, a two-dimensional program developed by Dr. J. Meek at the University of Queensland in Brisbane. Several modifications were incorporated by the writer in the program to reduce the data into a more meaningful format and to permit computer-plotting of principal stress and displacement vectors. Numerical modelling was conducted on the UNIVAC 1100 system at the Mount Isa Mine.
Stress input for the "probability of failure analysis" are determined by the boundary element program BITEM developed by Crotty and Wardle (1978) at the CSIRO Division of Applied Geomechanics. A simple computer program was written to expedite the analysis. This aspect of the work was completed on the CYBER 7600 system at the CSIRO.
CHAPTER 2 - MODEL FOR INTACT ROCK COMPONENT OF ROCK MASS

The mechanical properties of the intact rock component of the dolomitic shales are reviewed in this chapter. The work discussed below is based entirely on laboratory testing undertaken by Mount Isa Mine personnel and other researchers over the last 15 years. The writer's involvement is restricted to collation and assessment of the available data.

2.1 Strength of the Intact Rock

An extensive laboratory testing programme, involving several thousand core specimens, was undertaken between 1965 and 1978. The results of some of the tests have been reported in Rosengren (1968), MacLeod-Carey (1969), Mathews and Edwards (1969), Baczynski (1973, 1977c), Leask (1977) and Miller (1978).

2.1.1 Unconfined Compressive Strength

Both MacLeod-Carey (1969) and Mathews and Edwards (1969) note that the unconfined compressive strength is anisotropic and is dependent on the angular relationship between core axis and sedimentary layering (i.e., bedding planes).
Although there is some disagreement in regard to the absolute magnitude of the values, the indicated strength in directions parallel, normal and at 25-30° to the bedding planes is about 200-210 MPa, 300-360 MPa and 125-200 MPa, respectively. The standard deviation in each case appears to be approximately 35 MPa.

Brady (1977) suggested that the average strength of mineralized shale in an area of trial stoping within Orebody No.7 was 170 MPa, whereas the assessment of rock strength undertaken by Leask (1977) indicated a mean value of 188 MPa for the dolomitic shales. The latter work was restricted to evaluation of cores from the 1100 Orebody (copper) area of the mine, but had included samples from rock types which are typical of those in the silver-lead-zinc areas of the mine.

Figure 5 summarizes the results of core tests reported in Baczynski (1977c). The majority of test samples were obtained from diamond drill cores located within Orebody No.5 stratigraphic horizon at 14 Level of the mine.

The following trends may be deduced from Figure 5:

1. The unconfined compressive strength is markedly controlled by the angle between the core axis and sedimentary layering (i.e., bedding planes) of the mine shales. This angle is referred to as the "core-bedding angle".

2. The highest strengths are observed in specimens with core-bedding angles near 90° and the lowest values are recorded for specimens with angles near 30°.

3. For specimens failing by shear through intact rock, the strength of barren shale is greater than that of mineralized shale, for all specimens with core-bedding angles between 30° and 90°. At lower angles, specimens of each rock type appear to have similar strengths.
FIGURE 5. - Relationship Between Core-Bedding Angle and the Unconfined Compressive Strength of Cores. 
(4) For specimens failing along potential, pre-existing, weakness planes such as bedding or calcite infilled veins, the strength of both the barren and mineralized shale specimens is similar in the range of core-bedding angles between 0° and 60°. At greater angles, the strength of specimens appears to tend towards that displayed by specimens where failure is by shear through intact rock (for each respective rock type).

(5) Where a large sample of data is available, the strength values appear to be normally distributed about the mean value. An example of a distribution is presented in Figure 6. This observation is at variance with the trends reported by Torrent (1978), where a lognormal model is suggested for strength data.

(6) The average strength of barren and mineralized shale failing by shear through intact rock is estimated as 253 MPa and 221 MPa, respectively. These estimates are derived by partitioning data into six sub-groups. Grouping intervals correspond to ranges of core-bedding angles equal to 0-15°, 16-30°, 31-45°, 46-60°, 61-75° and 76-90°, respectively. The average value is determined by equal weighting of each subgroup mean.

(7) The average strength of barren and mineralized shale failing along a potential weakness plane in the cores is 139 MPa and 128 MPa, respectively. The same weighting process as described above was also applied to data.

The above results suggest that the unconfined compressive strength of the dolomitic shales is locally variable. A standard deviation of about 40-50 per cent of the mean value is commonly observed for each core-bedding angle. Strength is also dependent on the angle that bedding makes with the core axis of the test specimen.

A weighted average deduced for all the data plotted in Figure 5,
FIGURE 6. - STRENGTH OF 61 SPECIMENS OF MINERALIZED SHALE WITH ANGLE OF 31° - 45° BETWEEN LOADING AXIS AND BEDDING PLANE
including specimens failing along potential weakness planes in the cores, is 185 MPa with a standard deviation of 83 MPa.

2.1.2 Tensile Strength

Published information is restricted to that of MacLeod-Carey (1969), Mathews and Edwards (1969) and Mathews (1970). The results are based mainly on Brazilian tests. A diametral compression is applied between the platens to a rock cylinder. Tensile cracks parallel loading axis. Again, there are differences between the various sources.

The mean tensile strengths for directions of loading normal and parallel to bedding are indicated as:

\[
\begin{align*}
30 \text{ MPa and } 6 \text{ MPa,} \\
38 \text{ MPa and } 9 \text{ MPa,} \\
27 \text{ MPa and } 9 \text{ MPa,}
\end{align*}
\]

for the three sources cited above, respectively.

On the basis of these results, the average tensile strength in the two directions of loading is calculated as 32 MPa and 8 MPa, respectively. The overall mean tensile strength is 20 MPa.

2.1.3 Shear Strength, \( S_0 \), and Angle of Internal Friction, \( \phi_1 \)

The following parameters are determined with respect to shear failure
through intact rock specimens.

A diversity of terminology appears in technical literature to describe the two parameters. The definition used in this thesis is derived from Obert and Duvall (1967).

With respect to the Mohr's circle for stress, the shear strength $S_0$ of the rock refers to the intercept of Mohr's failure envelope on the shear stress axis of the plot. Angle of internal friction $\phi$ refers to the angular relationship between Mohr's failure envelope and the normal stress axis of the plot.

Assessment of the two parameters appears to be restricted to results reported in MacLeod-Carey (1969) and Miller (1978). A related study was undertaken by Rosengren (1968) where triaxial tests were conducted on cores with a range of core-bedding angles favourable to failure along bedding planes. This latter work yielded results on the shear characteristics of bedding planes and not that for shear failure through intact rock.

The results of MacLeod-Carey (1969) suggest a shear strength of about 30-35 MPa and an angle of internal friction at levels of normal stress within the range of stresses anticipated at the mine of about 55-60°. The corresponding values suggested by Miller (1978) are 18 MPa and 54°, respectively.

An alternative approach for the determination of the two parameters has been suggested by Wuerker (1959). The shear strength $S_0$ is deduced from a consideration of Mohr's representation of the unconfined tensile and compressive strengths. On the basis of the mean values of 20 MPa and 185 MPa deduced for the two strengths, this technique yields estimates of 34 MPa and 55° for shear strength and the angle of internal friction, respectively.

Horibe (1970) suggested that the shear strength intercept may be determined by the following formula:

$$S_0 = \frac{(\sigma_c - \sigma_t)\sqrt{\sigma_t}}{2\sqrt{\sigma_t} (\sigma_c - \sigma_t)}$$
where,

\[
\begin{align*}
S_0 &= \text{shear strength} \\
\sigma_c &= \text{unconfined compressive strength} \\
\sigma_t &= \text{unconfined tensile strength}
\end{align*}
\]

Utilizing the same mean values for the compressive and tensile strengths, the above equation yields a shear strength value of 38 MPa.

Overall, there is close agreement between various sets of estimates. A shear strength of 34 MPa and an angle of internal friction of 55° are assumed to be representative of the dolomitic shales at the mine.

The range of in-situ confining stresses is unlikely to exceed 60-80 MPa at the mine. Since these stresses are considerably lower than the average unconfined compressive strength of the shales, a significant variation in the operative angle of internal friction is not anticipated.

2.2 Modulus of Elasticity

Results of modulus determinations have been reported in MacLeod-Carey (1969), Mathews and Edwards (1969), Mathews (1970), MacKavanagh and Lee (1977) and Leask (1977).

On the basis of these, the mean modulus parallel and normal to bedding is estimated as 104 GPa and 91 GPa, respectively. The overall mean value for the static modulus of the intact rock is 100 GPa.
2.3 Poisson's Ratio

Results reported in the references cited in the previous paragraph indicate that the average values of Poisson's ratio in directions parallel and normal to bedding are 0.30 and 0.24. An overall mean value for the shales is 0.27.

2.4 Strain at Failure

On the basis of the unconfined compressive strength and modulus values for shale specimens with the bedding plane anisotropy normal and parallel to the direction of loading, the corresponding strain at failure in the two directions is 0.30 and 0.37 per cent, respectively. A value of 0.19 per cent is estimated when the overall mean compressive strength of all shale test specimens is considered.

Thus, failure of the intact rock may be anticipated to occur at strains of 0.2 - 0.4 per cent.

2.5 Summary

The mean material properties determined for the intact rock component of the rock mass system are summarized below.
Unconfined compressive strength = 185 MPa.
Tensile strength = 20 MPa.
Cohesion = 34 MPa.
Angle of Internal Friction = 55°
Static modulus of elasticity = 100 GPa.
Poisson's Ratio = 0.27
Strain at Failure = 0.3 per cent.
Several types of geological discontinuities are present within the dolomitic shales at the mine.

The aim of this chapter is to describe the physical characteristics of some of the fracture sets. Orientation, spacing, continuity and frictional properties are examined. Most of these parameters are included in the model for structural variability within the shales.

Chapter 4 supplements this work. It develops a model to account for the observed spatial distribution of fracture sets within the rock mass.

3.1 Source of Data

The information described below is largely based on the results of investigations that were undertaken at the mine during the early 1970's. A summary of this work has been reported in Baczynski (1974) and Bridges (1975). The former was concerned with the silver-lead-zinc orebodies. The latter concentrated on the development of a conceptual model for structure within the copper orebodies at the mine.

Some additional parameters have also been derived during field investigation undertaken as part of the present study.

Information on major structural features was collected by systematically mapping all accessible underground openings on several successive
levels at the mine.

Detailed investigations were undertaken at selected locations. Data was collected by means of line and area sampling of openings, as well as, by analysis of orientated diamond drill cores.

The line sampling method is a procedure whereby all fractures intercepted by a continuous, straight sample line are included in the data. The principal advantage of this technique is that it tends to yield unbiased results. It also permits a direct comparison to be made between in-situ mapping and orientated drill core data.

Area mapping entails the recording of all fractures that occur within a designated area. A more detailed discussion of this technique is presented in the next chapter. The underlying principles of the method are also reported in Baczynski (1980f).

3.2 Scope of Model

The most frequently observed geological weakness planes within the mine shales are "bedding plane partings" and "extension fractures" (more commonly termed joints). The latter refers to that group of discontinuities which are not bedding plane partings (i.e., not parallel to the sedimentary layering within the shales) and along which displacement of the two complementary rock surfaces has been normal to the fracture plane.

Locally, the shales are transected by major "shear fractures". Shear fractures are defined as those discontinuities with some relative displacement between the two complementary rock surfaces of the fracture plane. These geological structures are commonly referred
to as "faults". Transverse faults strike across the general strike of stratigraphy whereas strike faults trend parallel or semiparallel to stratigraphy.

Several different sets of shear fracture orientations have been recognised in the mine shales (Baczynski, 1974, 1977c). In areas where such shear fractures constitute major zones of weakness within the mass and where their continuity is proportional to or exceeds the dimensions of underground excavations, ground behaviour is largely governed by their presence.

In general, the occurrence of major shear fractures is restricted to well defined zones. Thus, their presence only affects the stability of relatively few stopes within the silver-lead-zinc orebodies at the mine. In view of this,

MAJOR SHEAR FRACTURES ARE NOT CONSIDERED IN THE PRESENT STUDY.

The scope of the model is limited to bedding plane partings, extension fractures and two sets of shear fractures (called Set Nos. 3 and 4 in this thesis) with continuities similar to those of the extension fractures.
Orientation refers to the attitude of weakness planes with respect to the x-, y- and z-coordinate axes.

Bedding plane partings constitute the most common plane of weakness in the shales. Three sets of fractures are also evident throughout the areas investigated. Other fracture orientations may attain local prominence.

The mean orientation of the various sets is not persistent. It appears to be locally variable and a considerable overlap between the sets is evident.

For purposes of the present study, geological structure within the dolomitic shales is defined in terms of:

**TEN DIFFERENT SETS OF FRACTURES AND A SET OF BEDDING PLANE PARTINGS.**

The range of orientations for each set is represented on a lower hemisphere, equal area, stereographic projection plot of poles to weakness planes in Figure 7.

The mean orientation of each set is indicated in Table 1.

Rigorous statistical methods have been suggested by Kiraly (1969), McMahon (1967), Markland (1974), Bridges (1975) and others for the definition and evaluation of fracture set orientation in terms of circular or elliptical normal probability density functions. However,
FIGURE 7.

Principal Fracture Sets
**TABLE 1.**

Characteristics of Fracture Sets

<table>
<thead>
<tr>
<th>Fracture Set No.</th>
<th>Mean Orientation*</th>
<th>Spacing(m)#</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>000/80</td>
<td>0.46</td>
</tr>
<tr>
<td>2</td>
<td>090/26</td>
<td>0.46</td>
</tr>
<tr>
<td>3</td>
<td>075/60</td>
<td>0.89</td>
</tr>
<tr>
<td>4</td>
<td>079/81</td>
<td>4.07</td>
</tr>
<tr>
<td>5</td>
<td>040/75</td>
<td>8.30</td>
</tr>
<tr>
<td>6</td>
<td>030/50</td>
<td>3.52</td>
</tr>
<tr>
<td>7</td>
<td>130/75</td>
<td>21.30</td>
</tr>
<tr>
<td>8</td>
<td>140/52</td>
<td>4.26</td>
</tr>
<tr>
<td>9</td>
<td>180/48</td>
<td>6.72</td>
</tr>
<tr>
<td>10</td>
<td>000/48</td>
<td>8.30</td>
</tr>
<tr>
<td>11 (5 O/B Hanging-wall)</td>
<td>270/65</td>
<td>0.17</td>
</tr>
<tr>
<td>11 (5 O/B Orezone)</td>
<td>270/65</td>
<td>0.34</td>
</tr>
<tr>
<td>11 (7 O/B Hanging-wall)</td>
<td>270/65</td>
<td>0.19</td>
</tr>
<tr>
<td>11 (7 O/B Orezone)</td>
<td>270/65</td>
<td>0.34</td>
</tr>
<tr>
<td>11 (11 O/B Hangingwall)</td>
<td>270/65</td>
<td>0.06</td>
</tr>
<tr>
<td>11 (11 O/B Orezone)</td>
<td>270/65</td>
<td>0.22</td>
</tr>
</tbody>
</table>

* Dip Direction / Dip Angle

# Derived from Line Sample Data
such an approach is not considered to be practical in terms of the present study.

ALL FRACTURES OR MEMBERS BELONGING TO A PARTICULAR SET ARE DESIGNATED BY A SINGLE MEAN ORIENTATION.

This model is similar to that adopted by Robertson (1970) for a "design joint"; a joint which has the properties which are average properties for a particular joint set.

3.3.2 Spacing

Spacing between discontinuities of the same set is the normal distance between successive fracture plane intersections along a straight line sample.

Studies at the mine suggest that the spacing between fractures of the same set can be best described by a lognormal probability density function. Frequency distributions for Set Nos. 2 and 3 are plotted on logarithmic probability paper in Figure 8. The illustrated relationships are based on diamond drill core data. Accordingly, this sample includes all fractures with continuities down to a lower limit of about 0.02 metre.

A lognormal model for fracture spacing has been deduced for each of the sets, including bedding plane partings.

The frequency of bedding plane partings per metre of stratigraphy has been recorded at some 150 locations within the immediate hanging-wall of the silver-lead-zinc orebodies. These measurements suggest
Cumulative Frequency Plot on Logarithmic Probability Paper for the Spacing of Fracture Sets 2 and 3 (Diamond Drill Core Data).

FIGURE 8.
a strong correlation between observed frequencies and the stratigraphic position of the orebody within the Urquhart Shales. A detailed discussion and summary plots are presented in Baczynski (1974).

A number of researchers have noted a relationship between the frequency of extension fractures and bed thickness. Harris et al. (1960) observed that for a given lithological type, the concentration of fractures was inversely related to the thickness of the bed. Price (1966) cites other studies which have yielded similar trends.

An investigation undertaken on approximately 400 metres of suitably orientated drill cores failed to establish a similar relationship between the frequency of bedding plane partings and other fracture sets within the mine shales. In contrast, this work indicates that the mean and median fracture frequency remains fairly consistent with increasing frequency of bedding plane partings. The median trend is presented in Figure 9. The scatter of results observed at bedding plane parting frequencies above 15 per metre arises from the lack of sample data in that region. The scatter of values is similar to that observed at lower bedding frequencies.

The main implications of this observation are that similar ranges of extension fracture frequencies are likely to occur throughout the mine shales, irrespective of the stratigraphic location of the orebody hanging-wall. Interpreted statistically, it means that:

THE OCCURRENCE OF BEDDING PLANE PARTINGS AND OTHER FRACTURE SETS IS INDEPENDENT.

Statistical testing confirms that there is no difference at the 5% level of significance between mean fracture frequency per metre and increasing frequency of bedding plane partings. As a result of an inadequate sample size, the hypothesis could not be tested for bedding plane parting frequencies above 10 per metre.
Relationship between Median Frequency of Extension Fractures and Frequency of Bedding Plane Partings.

FIGURE 9.
3.3.3 **Shape and Continuity**

Continuity defines the extent or size of geological discontinuities.

It is apparent that unless the fractures have a circular shape, such as has been assumed by Secor (1968), Robertson (1970) and others, the measured continuity will be dependent upon:

1. the chord of intersection of the fracture plane with the sample area, and

2. the orientation of the sample area with respect to that of the major and minor axes defining the fracture plane.

The determination of the two dimensional shape of fractures is a difficult task. It requires that the extent of individual fractures should be observed in the plane of the fracture. Such observations are rare, especially for the more continuous fractures within a rock mass. Thus, one is forced to adopt a compromise approach and to estimate statistically the continuity distribution in several directions. In practice, geometry of mine layouts further restricts directions of observation. Commonly, it is only possible to observe fracture traces in two, mutually perpendicular, directions.

Measurement of fracture continuities is also constricted by the finite dimensions of the area available for observations. It is impossible to observe the true continuity of vertically or near-vertically dipping fractures whose continuity exceeds the height of underground openings. One or both extremities of such planes may be located outside the field of view. This problem is well recognised, e.g., Piteau (1973), Cruden (1977) and others.

Thus, one is restricted to the study of horizontal or near-horizontal fracture planes that may be observed in mutually perpendicular underground openings such as drives and cross-cuts within the mine.
On the basis of such a study undertaken for fracture Set No.2, it has been found that:

(1) fracture continuity parallel and normal to bedding in the mine can be best approximated by lognormal probability density functions,

(2) the average ratio of continuities in the two directions is approximately 1.0 : 0.44, respectively, and

(3) the minimum continuity of fractures in the direction parallel to bedding is about 0.1 metre.

The above results are presented graphically on logarithmic probability paper in Figure 10.

Although the results may be fortuitous, Skempton et al. (1969) suggested a similar ratio of major and minor axes for continuity of fractures.

On the basis of the rather limited study, a rectangular shape with some possible minor rounding of corners is tentatively postulated for the average fracture plane. The major axis of this average plane trends parallel to bedding.

A similar shape and continuity model is assumed for fracture Set Nos. 1 - 10.

The continuity of bedding plane partings (Set No. 11) also appears to be in accord with a lognormal model. The observed frequency distributions are presented on logarithmic probability paper in Figure 11.

Although rigorous field evidence is not available, the writer is of the opinion that the shape of the average bedding plane parting is probably equidimensional.
Cumulative Frequency Plot on Logarithmic Probability Paper for Continuity of Fracture Set No. 2 in Directions Parallel and Normal to Bedding of Mine Shales (Line Sample Data).

FIGURE 10.
Cumulative Frequency Plot on Logarithmic Probability Paper for Continuity of Bedding Plane Partings (Line Sample Data).

FIGURE 11.
3.3.4 Frictional Properties of Fractures

A major study of frictional properties of fractures within the shales was undertaken by Rosengren (1968). Less extensive work is reported in Herget (1968) and Mathews (1970).

Frictional characteristics of fractures are essentially governed by:

1. the planarity of their surfaces,
2. nature of the wall rock and the degree of interlock between the surfaces,
3. nature of the infilling materials.

On the scale of 54 mm diameter core samples, the majority of fracture surfaces in Set Nos. 1 - 10 appear to be relatively planar, but minor undulations and steps (1-5 mm in depth) are locally apparent (Baczynski, 1974).

However, field investigations suggest that only a few of the more continuous fracture planes (say, greater than 3 metres) are likely to be completely planar.

A high degree of interlock is observed between fracture plane surfaces.

About 60 per cent of the fractures display clean surfaces without any infilling materials along them. The remaining fractures are infilled with limonite, carbonate, pyrite, chlorite or graphite. Combinations of the above minerals are also apparent along some planes.

Tests conducted by Rosengren (1968) on small diameter cores suggest that the residual angle of friction varies from about 40° for fresh, clean surfaces to about 10° for some of the polished, graphitic surfaces. Cohesion ranges from 0.0 to 1.4 MPa.
On the basis of:

(1) the relative frequency of infilling material types observed in a sample of 1440 fracture surfaces (Baczynski, 1977c), and

(2) the frictional properties derived by Rosengren (1968) for various types of fracture surfaces,

a mean residual angle of friction of $32^\circ$ and a cohesion of 0.8 MPa is suggested for the average fracture plane of Set Nos. 1 - 10. Furthermore, the relationship noted by Mathews (1970) between fracture roughness and peak friction angle suggests that this angle for the average fracture plane may be about $36^\circ$ to $40^\circ$.

The majority of bedding plane partings (Set No. 11) are relatively planar on the local scale. However, 0.5 - 2.0 mm deep slickensides, major and minor undulations, stepped offsets of 1 - 20 mm (caused by cross fractures) and other surface irregularities are commonly apparent on the more continuous planes. Complete to almost complete interlock is observed between most surfaces.

Bedding plane partings may be classed as:

(1) clean-fresh,

(2) polished and graphitic.

An analysis of 1000 surfaces intercepted by diamond drilling suggests that 43 per cent of the planes are in the second class. Results of this study are summarized graphically in Figure 12.

In general, the proportion of graphitic bedding plane surfaces within a sample size of "n" observations may be described statistically by a normal probability density function with a mean of 0.43 and a variance of 0.73/n.
Cumulative Frequency Distribution on Normal Probability Paper for Proportion of Graphitic Bedding Plane Partings in Samples of 10 Partings (Total Sample Size = 1000).

FIGURE 12.
Available data suggests that the residual angles of friction for smooth, graphitic and non-graphitic planes are about $10^\circ - 18^\circ$ and $32^\circ - 34^\circ$, respectively. This angle approaches $45^\circ$ for relatively rough surfaces, irrespective of surface coating.

Cohesion and peak friction angles of bedding plane partings are similar to those noted for fracture Set Nos. 1 - 10.
One of the parameters affecting rock mass strength and stability is its inherent blockiness.

The degree of blockiness is largely a function of the number of fracture sets and their respective orientations, continuities and intensities within the mass. Progressive ravelling or fretting away of rock surfaces is commonly observed in mine areas which are transected by several sets of closely spaced fractures. It is also difficult to install effective ground support in such areas. Thus, the assessment of local variability in fracture intensities constitutes an important component in the development of a model for geological structure.

Fracture intensities within a mass are a function of the spatial distribution of the various sets of discontinuities within it.

**SPATIAL DISTRIBUTION REFERS TO THE POSITION OF FRACTURE PLANE CENTRES WITHIN A DEFINED VOLUME OF ROCK MASS.**

The aim of the present chapter is to examine and to test the validity of various alternate hypotheses for spatial distribution of fractures within rocks. Aspects covered include:

1. Formulation of a model to account for the observed distribution of various fracture sets within the dolomitic shales.

2. Definition of a field mapping procedure for collection of the necessary data.

3. Validation of the proposed spatial model by computer simulation of fracture plane distribution within rock mass blocks.
4.1 General Comments

A literature survey indicates that very little information has been published on this topic. A brief review of the subject is presented in Baczynski (1978b). The four alternate spatial models that have been suggested are:

(1) Even
(2) Variable
(3) Random
(4) Zonal

4.2 "Even" and "Variable" Models

The "even" spacing model has been suggested by some writers, e.g., Hodgson (1961) and Turner and Weiss (1963). However, the lognormal model derived for spacing between adjacent fracture of a set within the mine shales indicates that fracture clustering occurs. This evidence militates against the "even" spacing hypothesis.

A "variable" spacing model has been postulated by others, e.g., Nickelsen and Hough (1967) and Moseley (1968). However, such models are only of a qualitative and descriptive nature.
4.3 "Random" Model

Although not explicitly stated, most geotechnical analyses assume a random model for spatial distribution of various fracture sets. The concept of random distribution is also implicitly supported by Bridges (1975).

The validity of the model can be most readily tested with the aid of a simple computer program based on the "Monte Carlo" method. Aspects of "Monte Carlo" simulation are described in Hammersley and Handscomb (1964).

The necessary input data for each set of fractures to be simulated within a two-dimensional plane consists of:

1. average orientation of the fracture traces to be assigned to each member of the set,
2. a model for continuity of fracture traces,
3. mean spacing between members of the set as noted along line samples, and
4. a suitable model for the number of fractures or the total length of traces to be generated within the defined area.

The first three parameters may be derived from existing line sample data. The last requires some consideration. The various methods used to generate random fracture distributions and their degree of success are discussed in Baczynski (1977a). The procedure adopted is based on the following conceptual considerations.

Assume that the physical dimensions of the area designated for fracture trace generation are very large with respect to the mean
spacing between fractures. Then, on the average, it may be expected
that the mean spacing between traces encountered along any hypothet­
al sample line traversing this area in the direction normal to
fracture traces will approach the mean value for the set. This
condition should be satisfied irrespective of the model used for
the spatial distribution of fracture plane traces within the
designated area. Any number of parallel sample lines can be con­
sidered. In the limit, sample line spacing may be assumed to be
infinitely close.

On this basis, the total trace length of all fractures within the
generation area may be simply estimated by the formula:

\[ T = \left( \frac{D_n}{S_m} \right) \times D_p \]

where,

- \( T \) = total trace length,
- \( D_n \) = average dimension of the
generation area in direction
  normal to fracture traces,
- \( D_p \) = as above, but parallel to
  fracture traces, and
- \( S_m \) = mean spacing between fractures
  of the set along sample lines.

This estimate of the anticipated total trace length is used to
furnish the fourth parameter.

A relatively simple computer program was developed by the writer to
test the validity of the "random" model. The basic procedure adopted
for the generation of fracture traces within a designated area con­
sisted of the following three iterative steps:

1. Random generation of mid-point coordinates for fracture
   trace.

2. Statistical generation of a trace length in accordance
with the fracture continuity model established for the mine shales.

(3) Determination of x- and y-coordinates for the extremities of the fracture trace through the designated midpoint.

The iterative procedure is terminated when the cumulative trace length of generated fractures equals or just exceeds the permissible total length.

4.4 Testing of "Random" Model

The coordinate data generated by the above computer program was output to a drum plotter and the results presented in the form of fracture traces. The resulting patterns were tested by computer line sampling and a spacing model deduced.

Figure 13 presents a typical fracture trace pattern generated on the basis of the "random" model. The most striking feature of this plot is that:

THE DENSITY OF FRACTURES IS RELATIVELY HOMOGENEOUS OVER THE ENTIRE AREA.

The resulting cumulative frequency plot for spacing between adjacent fractures is presented in Figure 14. It is readily apparent from this plot that:

THE "RANDOM" MODEL FAILS TO REPRODUCE THE LOGNORMAL SPACING DISTRIBUTION OBSERVED AT THE MINE.
Pattern of Fracture Traces Generated on Basis of "Random" Model for Spatial Distribution of Fracture Plane Centres.

FIGURE 13.

FIGURE 14.
On the basis of the above results, the validity of the "random" model is dismissed for the spatial distribution of fractures within the dolomitic shales. It is apparent that this model underestimates the extent of local variability in the intensity of fracturing at the mine.

4.5 Basis of "Zonal" Model

A "zonal" model for spatial distribution of fractures has been postulated by several workers over the last 20 years. In all instances, the model was suggested intuitively on the basis of general field observations relating to fracture clustering. It was presented without the firm evidence that is necessary to disprove the commonly stated converse hypothesis that the observed spatial relationships may have originated as a result of a "random" process.

The evidence cited in the section above demonstrates that the "random" model fails to account for the spatial relationship observed at the mine. This has necessitated the formulation of an alternative hypothesis which has been termed by the writer as the "zonal" model.
4.6 Derivation of "Zonal" Model Parameters

4.6.1 Field Investigations

The structural model derived at Mount Isa during the early 1970's is based on a single line sampling procedure. This data was found to be inadequate for formulation of two- and three-dimensional models for distribution of fracture clusters. Further field investigations were necessary.

The additional work centred on mapping of continuous area traverses at several location in the mine. The horizontal length of uninterrupted traverses ranged from 20 metres to 70 metres. Field of vertical observation was between 3 metres and 5 metres. Site investigations were restricted to fracture Set Nos. 1 - 10.

A simple and rapid procedure was adopted for mapping of underground openings.

Selected underground openings were photographed. The resulting, suitably enlarged, overlapping photographs served as base maps to record the traces of all fractures observed on the walls of the openings. On completion of underground mapping, the data was transferred to non-distorted maps and fracture traces were assigned to "sets" on the basis of their orientation. A separate transparent overlay was compiled for each set.
4.6.2 Evaluation of Data

"Unit Areas" are defined on each transparent overlay in the manner illustrated in Figure 15. For purposes of the present analysis:

A UNIT AREA IS DEFINED AS AN AREA EQUIVALENT TO 1.0 METRE SQUARED, MEASURED ON A PLANE PERPENDICULAR TO THE AVERAGE ORIENTATION OF THE FRACTURE SET, AND WITH ITS SIDES PARALLEL AND PERPENDICULAR TO THE STRIKE OF THE SET.

This permits the dimensions of unit areas to be adjusted according to the angular relationship between the strike of the set and the strike of the underground opening that is being evaluated. In this manner, the "normalized" unit area remains the same or constant for each set.

The prime objective of this analysis has been to derive a model for local variability in the intensity of fracturing between unit areas.

The total trace length of each set of fractures was determined within each unit area. Conditional probability density functions were then derived to describe the extent of fracture intensity "zones" in directions normal and parallel to the average trace of each set. The conceptual basis is illustrated in Figure 16.

The following generalization is necessary for a 3-D model:

EXTRAPOLATION OF THE 2-D MODEL INTO THE THIRD DIMENSION ASSUMES THAT THE INTENSITY OF FRACTURING REMAINS HOMOGENEOUS FOR A UNIT DISTANCE INTO THE THIRD DIMENSION.

For example, a two-dimensional unit area with an intensity of 8.0 metres of fracture traces of a set is converted in the three-dimensional
A = Central Unit Area (or Unit Volume) with respect to B and C.
B,C = "Up Dip" and "Down Dip" Unit Areas with respect to A.
D = "Zero Intensity" Unit Area.
E = "Non-zero Intensity" Unit Area.
D,E = "Adjacent" Unit Areas with respect to C.

Unit Areas with respect to a Particular Set.

FIGURE 15.
FIGURE 16. - Conceptual Relationship: Influence of Immediately Adjacent Areas on Observed Fracture Intensity Within a Designated Unit Area.
model to 8.0 square metres of fracturing per cubic metre or unit domain.

It is apparent that such extrapolation would be very difficult to justify where relatively large dimensions have been selected for unit areas or unit volumes. The smaller the dimensions, the more likely it is that the assumptions are valid. The dimensions of unit areas selected for the mine shales are approximately 2.5 times the mean spacing between fracture planes of the most common set (for set Nos. 1 - 10) and about 0.7 times their mean trace length.

4.7 Some Statistical Considerations

The objective of the analysis is to formulate a statistical model to account for the observed extent of fracture "zones" in directions parallel and normal to the average orientation of fracture planes in each set. In a two-dimensional, local coordinate system, these directions of influence correspond to the y-axis and the x-axis, respectively.

One of the considerations in processing of field data is to examine various avenues of transforming the results into a form more amenable to standard statistical analysis. Several statistical models are available.

The normal probability density function is undoubtedly the most frequently used of all probability laws (Larson, 1974). This arises because:

(1) the normal random variable occurs frequently in practical problems,
Another continuous function useful in many geological situations is the lognormal density (Krumbein and Graybill, 1965). In its simplest form, the lognormal distribution is defined as the distribution of a variate whose logarithm obeys the normal law of probability (Aitchison and Brown, 1976). Thus, if a transformation is made, such that "L" is equal to Log b X (where "b" is the base of the logarithm), the random variable "L" is a normal variable.

Care should be always exercised to ensure that standard distributions are not assumed where these exhibit a poor degree of correlation with field data. However, results are in a more manageable form if such approximations are possible. The computational effort involved in model testing is also considerably reduced.

Consider a hypothetical example.

Suppose that the intensity of a particular fracture set occurring within a unit area is influenced by the set's intensities in the immediately adjacent areas. Assumed directions of influence are both parallel and normal to the average fracture trace of the set. The influence in direction parallel to the average fracture trace is described by a normal probability density function with mean \( \mu^i \) and standard deviation \( \sigma^i \). Values of the two parameters are conditional on "i"; the actual intensity observed in the adjacent unit area within the specified direction of influence. Similarly, the influence in direction normal to the average fracture trace is described by a normal probability density function with mean \( \mu^j \) and standard deviation \( \sigma^j \). Again, values of the two parameters are conditional on "j"; the actual intensity observed in the adjacent unit area within the specified direction of influence.

The fracture intensity within any unit area may then be expressed statistically by a joint probability density function of the two continuous random variables \( X^j \) and \( Y^i \), such as for example illustrated.
Given that the intensities in the two adjacent areas are "i" and "j", respectively, then the resulting joint function may be written as:

\[
 f_{x^j,y^i}(x^j, y^i) = \frac{1}{\sqrt{2\pi} \sigma_x^j} \cdot \exp \left\{ -\frac{1}{2} \left( \frac{x^j - \mu_x^j}{\sigma_x^j} \right)^2 \right\} \cdot \frac{1}{\sqrt{2\pi} \sigma_y^i \sqrt{1 - \rho^2}} \exp \left\{ -\frac{1}{2} \left( \frac{y^i - \mu_y^i - \rho \left( \frac{\sigma_y^i}{\sigma_x^j} \right) \cdot (x^j - \mu_x^j)}{\sigma_y^i \sqrt{1 - \rho^2}} \right)^2 \right\}
\]

where,

\[
 \mu_x^j, \mu_y^i, \sigma_x^j \text{ and } \sigma_y^i \text{ are as defined earlier, and}
\]

\[
 \rho = \text{correlation coefficient between } X^j \text{ and } Y^i,
\]

with values in the range \(-1 \leq \rho \leq 1\)

\[
 \text{Covariance} \left( X^j, Y^i \right) = \frac{\text{Expected Value of Joint Distribution } X^jY^i - \left( \text{Expected Value of } X^j \times \text{Expected Value of } Y^i \right)}{\sigma_x^j \cdot \sigma_y^i}
\]

The correlation coefficient is equal to zero for independent \(X^j\) and \(Y^i\) variables.

In terms of the present analysis, the random variables \(X^j\) and \(Y^i\) may be considered to be essentially independent. Both the \(f_{x^j}(x^j)\) and \(f_{y^i}(y^i)\) marginal probability density functions are derived independently. The probability that a particular fracture intensity will be encountered within an unit area may be readily determined by integration of the joint density function, where:

FIGURE 17.
In simple terms, the probability corresponds to the volume under the surface of the joint distribution \( f_{x^j, y^i}(x, y) \).

There is no doubt that characterization of fracture intensities in terms of such functions simplifies the model, provided that the assumed distributions are in reasonable accord with field evidence. However, it should be appreciated that it is commonly impossible, because of sampling limitations, to validate models in the tails or extreme ends of distributions. In fact, discontinuities with the characteristics inferred by the extreme values may not even exist in the prototype. A more logical approach in these circumstances may be to constrain the range of permissible values. The resulting model will be statistically "untidy", but it may be more representative of the real situation.

4.8 Spatial Relationship Observed at the Mine

In accordance with the above discussion, the results of the analysis have been transformed into a form that is more amenable to statistical analysis.

A variable INT is used to represent the intensity of fractures per unit area (i.e., metres of fracturing per square metre, m/m\(^2\)). It is an adjusted value derived from "Observed Field Intensity X" plus a constant value of 4.0 added. With this constant adjustment, the distribution of the random variable INT may be approximated by a lognormal probability density function. \( \log_{10} \) (INT) is a normal variate. A non-negativity constraint is implied logically because
negative fracture intensities are impossible. Also, on the basis of field observations, an upper constraint of 14.5 m/m² has been imposed on the range of values that may be assumed by INT. This consists of a maximum field value of 10.5 m/m² and the constant 4.0 m/m².

Typical field results are presented on logarithmic probability paper in Figures 18 - 20. The first plot summarizes local variability in fracture intensity between unit areas for Set No.1. The latter two indicate intensities observed immediately adjacent to unit areas with a known intensity "i". Results are presented for two, mutually perpendicular directions, i.e., parallel and normal to the average orientation of the particular fracture set.

The above cited plots demonstrate that fracture intensities are not random. They are markedly dependent on intensities which occur in adjacent areas. This dependence is most pronounced for the direction of influence parallel to the average orientation of the set.

Similar types of trends were observed for each of the other sets examined.

Three main conclusions are apparent on the basis of the available field data:

(1) DISTRIBUTION OF EACH SET OF FRACTURES IS NOT UNIFORM THROUGHOUT THE ROCK MASS.

The various defined sets, in particular the less common ones, are only present locally. Accordingly, the mass may be characterized in terms of "non-zero intensity" and "zero intensity" unit areas. These correspond to local occurrence and absence of a particular set, respectively. Where a set occurs, intensities may be described by a distribution similar to that presented in Figure 18. Allowable intensities are bounded by non-negativity and upper limit constraints.
Cumulative Frequency Plot on Logarithmic Probability Paper for Local Variability in the Intensity of Fracture Set No.1

FIGURE 18.
Probability Density Functions for Intensities of Fractures Observed Adjacent to Unit Areas with a Known Intensity "i" in Direction Parallel to Average Orientation of Set No. 1.

FIGURE 19.
Probability Density Functions for Intensities of Fractures Observed Adjacent to Unit Areas with a Known Intensity $i$ in Direction Normal to Average Orientation of Set No. 1.

FIGURE 20.
(2) **EXTENT OF "ZONES" IN THE DIRECTION PARALLEL TO THE AVERAGE ORIENTATION OF A PARTICULAR SET IS CONDITIONAL ON THE INTENSITY OBSERVED IN AN IMMEDIATELY ADJACENT UNIT AREA.**

The probability of encountering the same or greater intensity is about 0.5. Non-negativity and upper limit constraints apply.

(3) **EXTENT OR WIDTH OF "ZONES" IN THE DIRECTION NORMAL TO THE AVERAGE ORIENTATION OF A PARTICULAR SET IS LESS MARKEDLY INFLUENCED BY THE INTENSITY OBSERVED IN AN IMMEDIATELY ADJACENT UNIT AREA.**

Clear association is only evident for adjacent intensities equal to or less than 1.5 m/m². At higher intensities, the influence is less convincing and may be approximated by a single function, with little loss in accuracy.

In view of the latter observation, a simplified structural model has been assumed for the dolomitic shales. It presumes that the extent of "zones" in the direction normal to the average orientation of fracture planes is independent of the intensity occurring in the immediately adjacent unit area.

The model for local variability in the intensity of fracturing is expressed in terms of three types of probability functions for each of Set Nos. 1 - 10:

(1) A binomial distribution function for probability of fracture set occurrence within unit areas, i.e., probability of "non-zero intensity" unit areas. Probabilities for the first ten sets are 0.898, 0.898, 0.464, 0.102, 0.050, 0.117, 0.019, 0.097, 0.062 and 0.050, respectively.
(2) An adjusted and transformed, normal probability density function for determination of fracture set intensities within "non-zero intensity" unit areas. Probability distributions are similar to that presented in Figure 18. Permissible range of intensities is (before adjustment):

Set Nos. 1-3 = 0.1 - 10.5 m/m²
           4-10 = 0.1 - 6.5 m/m²

(3) A set of adjusted and transformed conditional probability density functions for extent of "zones" in direction parallel to the average orientation of a particular set. Probability distributions are similar to those presented in Figure 19. Permissible ranges of intensities (before constant adjustment) are:

Set Nos. 1-3 = 0.0 - 10.5 m/m²
           4-10 = 0.0 - 6.5 m/m²

Maximum allowable difference between any two adjacent unit areas is 4.0 m/m². The magnitude of this parameter is partially based on field observations. Computer simulation also indicated that intensities had to be constrained in order to reproduce overall mean intensities in accordance with field measurements.
4.9 Model Testing

The objective of the testing programme was to confirm that the intensity and spatial distribution of structural features within rock mass blocks simulated on the basis of the proposed "zonal" model are in accord with in situ observations. The validity was ascertained by an examination of the generated fracture intensities per unit domain and the resulting spatial relationships between adjacent fractures along sample lines.

A computer program CRACKS-2 was developed by the writer to assist in the simulation process.

The function of the program is to model fracture plane distributions within three-dimensional blocks. The program has the capacity to generate fracture clusters in accord with a given set of input parameters for the "zonal" model, to output the generated fracture patterns for specified sections through the block via a computer plotter, to line sample the ensuing fracture patterns and then to output the resulting statistical distributions for spacing between adjacent fractures in a number of graphical forms. The program has been subdivided into several independent sub-programs or modules. This approach was considered to be necessary to permit a more convenient manipulation of the potentially large volumes of data that may be generated statistically. A detailed description and a full listing of the program are included in Baczynski (1978a).

Six basic steps are employed in the program for the generation of each set of fracture planes within a test block:

1. Selection of dimensions for the test block and the determination of dimensions for a "master block". The master block refers to a minimum size, cubic block whose dimensions are such that the block will always fully enclose the test block, irrespective of the angular rotation about a common central axis that may be imposed on either of the two blocks. The concept is illustrated in Figure 21.
Conceptual Relationship Between Test-Block and Master-Block in Computer Program CRACKS-2.

FIGURE 21.
(2) Assignment of fracture intensities to each unit domain of the master block in accordance with the input statistical parameters for the "zonal" model.

(3) Generation of fracture planes with a two-dimensional extent in accordance with the fracture continuity model. The procedure is terminated when the total intensity assigned to the master block is satisfied.

(4) Sorting of fracture planes into a descending sequence of continuities (i.e., largest planes first) and location of individual planes within the master block. Clustering of fracture planes is in accordance with the intensities assigned to "zones" or groups of domains within the block. All fracture planes are initially orientated parallel to the y-z coordinate plane of the master block.

(5) Rotation of the master block about the central axis to permit each fracture plane contained within it to assume the average orientation for that set in space. Fracture planes are tested for intersection with the test block and a permanent computer file is created for all intersecting planes.

(6) Calculation of the mean fracture set intensity per unit domain of the master block and the generation of fracture patterns on selected planes transecting the test block. Determination of spacing model for adjacent fracture traces intercepted along specified line traverses.

On the basis of several hundred master blocks, each comprised of about 3500 unit domains, an acceptable level of correlation was achieved between the generated and in situ fracture intensity for each set. The results are summarized in Table 2.

A typical fracture pattern generated for set No.1 is illustrated in Figure 22 and the corresponding line sample spacing results are presented on logarithmic probability paper in Figure 23. Line sampling results for most of the other sets are included in Baczynski (1978b).
TABLE 2.

Mean Intensity of Fractures per Unit Volume

<table>
<thead>
<tr>
<th>Fracture Set No.</th>
<th>In-situ</th>
<th>Simulated</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.17</td>
<td>2.15</td>
</tr>
<tr>
<td>2</td>
<td>2.17</td>
<td>2.15</td>
</tr>
<tr>
<td>3</td>
<td>1.12</td>
<td>1.12</td>
</tr>
<tr>
<td>4</td>
<td>0.24</td>
<td>0.27</td>
</tr>
<tr>
<td>5</td>
<td>0.12</td>
<td>0.14</td>
</tr>
<tr>
<td>6</td>
<td>0.28</td>
<td>0.31</td>
</tr>
<tr>
<td>7</td>
<td>0.05</td>
<td>0.06</td>
</tr>
<tr>
<td>8</td>
<td>0.23</td>
<td>0.27</td>
</tr>
<tr>
<td>9</td>
<td>0.15</td>
<td>0.18</td>
</tr>
<tr>
<td>10</td>
<td>0.12</td>
<td>0.14</td>
</tr>
</tbody>
</table>

Results are expressed in $m^2/m^3$. 
Typical Fracture Pattern Generated for Fracture Set No. 1 on Basis of "Zonal" Model.

FIGURE 22.
Cumulative Frequency Distribution on Logarithmic Probability Paper for Spacing Between Adjacent Fractures of Set No.1 Derived on Basis of "Zonal" Spatial Model.

FIGURE 23.
SPACING BETWEEN ADJACENT FRACTURES (FOR EACH SET TESTED) DISPLAYS A VERY GOOD CORRELATION WITH A LOGNORMAL PROBABILITY DENSITY FUNCTION.

Fracture patterns generated on six successive planes through a test block with dimensions of 10m x 8m x 6m are illustrated in Figure 24. Although these plots are restricted to the first six sets listed in Table 2, rock mass variability and the contrast between low and high fracture intensity sub-domains are clearly evident in this figure.
FIGURE 24. - Fracture Pattern for Set Nos. 1 - 6 Generated on Six, 10m x 6m Planes Through a Test Block on the Basis of "Zonal" Model for Spatial Distribution (Depth of Planes into Test Block is Indicated Beside Each Pattern).
4.10 **Spatial Model for Bedding Plane Partings**

A detailed statistical study of the two-dimensional extent of bedding plane parting (Set No. 11) "zones" was not undertaken at the mine.

Investigations are largely restricted to the evaluation of the average intensity of the partings at various accessible locations in the immediate hanging-walls of the orebodies. Locally, observations have been supplemented by diamond drill core data.

This simple approach was necessitated by sampling considerations.

As noted earlier, the mean intensity of bedding plane partings appears to be stratigraphically controlled. There are marked differences between various orebody hanging-walls. Because of this control, the development of any realistic "zonal" model would have necessitated an area mapping programme within very specific stratigraphic horizons of the mine shales. Access to such locations is somewhat limited. Thus, the proposed investigations would have only yielded a small sample of data. This sample size would prove inadequate for the formulation of conditional probability functions similar to those that were derived for fracture Set Nos. 1 - 10.

THE ADOPTED MODEL FOR BEDDING PLANE PARTINGS IS BASED ON LOCAL VARIABILITY NOTED IN THE MEAN INTENSITY OF THE PARTINGS WITHIN OREBODY HANGING-WALLS.

The model is supplemented with general observations on the persistence of ground conditions within underground stopes.

The following model has been assumed:

(1) The average intensity of bedding plane partings is stratigraphically controlled.
(2) Although local variability is apparent, relatively homogeneous mean intensities may characterize "zones" or domains within particular stratigraphic horizons. These may be of considerable area in their two-dimensional extent.

(3) Local variability within any stratigraphic horizon may be described by a normal probability density function. The mean of the function is equal to the average intensity of bedding plane partings noted for the particular stratigraphic horizon. The standard deviation is equal to 50 per cent of this mean value. A probability plot for the intensity of bedding plane partings in the immediate hanging-wall of Orebody No. 11 is presented in Figure 25.

(4) Variability between segments of a stratigraphic horizon roughly equivalent in area to the dimensions of a typical hanging-wall of a sublevel open stope may also be approximated by a normal probability density function. The mean intensity is as defined above. The standard deviation is about 35 per cent of this mean value. A probability plot for Orebody No. 11 is presented in Figure 26.
Cumulative Frequency Plot on Normal Probability Paper for Local Variability in the Intensity (per metre) of Fracture Set No.11 (Bedding Plane Partings) in the Immediate Hanging-wall of Orebody No.11.

FIGURE 25.
Cumulative Frequency Plot on Normal Probability Paper for Variability in the Mean Intensity (per metre) of Fracture Set No.11 (Bedding Plane Partings) Between Stope Hanging-walls in Orebody No.11.

FIGURE 26.
A model for structural variability between rock mass blocks of unit volume (i.e., 1.0 m$^3$) was developed in the previous chapter. The purpose of the present chapter is to extend the model to blocks with larger dimensions. Potential variability in the average structural conditions within such blocks is examined.

5.1 Model for Each Fracture Set

Any large block of rock mass can be viewed as a collection of unit volumes. A block with a volume of 100 cubic metres is simply comprised of 100 unit volumes. Structural variability within the unit volumes is in accord with the statistical model developed in Chapter 4.

The model for structural variability between large blocks was developed in the following manner. For each selected block size, geological structure was generated by assigning and propagating fracture intensities within individual unit domains comprising the block in accordance with the "zonal" model. The average intensity of a particular set within a block was derived by:

$$\text{Average Intensity} = \frac{\sum_{i=1}^{n} I_i}{n}$$

where, $I_i$ = fracture set intensity assigned to the $i$-th unit domain,
and \( n \) = volume of block or number of unit domains within block.

Blocks with nine different dimensions were evaluated. Their dimensions ranged from 8 to 3375 cubic metres. Each block in the series was roughly twice the dimensions of the preceding one. The actual range of dimensions selected was restricted by limitations inherent in the computer program CRACKS-2 which was used for the simulation process.

A sample of 200 blocks was generated for each selected dimension and for each fracture set.

The resulting ranges of average intensities were approximated by normal probability density functions. The results displayed a good degree of correlation with the normal distribution for blocks comprised of 50 or more unit domains.

The results of the simulations are presented in Figure 27. The relationship between volume of the block and the "mean" and "standard deviation" of the normal probability density function describing variability in fracture intensity is indicated. For example, the range of intensities of fracture Set No. 1 in blocks comprised of 100 unit domains may be approximated by a normal function with a mean and standard deviation of \( 2.15m^2/m^3 \) and \( 0.55m^2/m^3 \), respectively.

It should be noted that negative fracture intensities are not possible. The cumulative area under the probability density function below zero represents the frequency of blocks in which the particular fracture does not occur.
Approximate Relationship Between Volume of Rock Mass & The Mean and Standard Deviation of Normal Probability Density Functions For Intensity of Fracture Set Nos.1-3 ($m^2/m^3$).

FIGURE 27.
Approximate Relationship Between Volume of Rock Mass & The Mean and Standard Deviation of Normal Probability Density Functions for Intensity of Fracture Set Nos. 4, 5 & 10 (m²/m³).

FIGURE 27 (Cont.)
Approximate Relationship Between Volume of Rock Mass & The Mean and Standard Deviation of Normal Probability Density Functions for Intensity of Fracture Set Nos. 6 and 7 ($m^2/m^3$).

FIGURE 27 (Cont.)
Approximate Relationship Between Volume of Rock Mass & The Mean and Standard Deviation of Normal Probability Density Functions for Intensity of Fracture Set Nos. 8 and 9 (m$^2$/m$^3$).

FIGURE 27 (Cont.)
Approximate Relationship Assumed Between Volume of Rock Mass & The Mean and Standard Deviation (Expressed as a Ratio of the Mean Value) of Normal Probability Density Functions for Intensity of Fracture Set No. 11 (Bedding Plane Partings) ($m^2/m^3$).

FIGURE 27 (Cont.)
5.2 Combined Model for Rock Mass

Variability in the average intensity of any fracture set within the mine shales may be approximated by means of normal probability density functions. Thus, the intensity may be considered as a normal variate, i.e., \( n(\mu_i, \sigma_i^2) \) - variate, where \( \mu_i \) and \( \sigma_i^2 \) are the mean and variance of the normal probability density function of the \( i \)-th fracture set.

The average intensity of fracturing of all the sets combined may be considered in terms of a multivariate probability density function of "n" random variables \( x_1, x_2, x_3, \ldots, x_n \), where each random variable refers to the average intensity of a particular set of fractures within the rock mass.

The multivariate probability function is some function:

\[
f_{x_1; x_2; x_3; \ldots, x_n}(x_1; x_2; x_3; \ldots, x_n)
\]

such that \( P(x_1 \leq a_1; x_2 \leq a_2; x_3 \leq a_3; \ldots; x_n \leq a_n) \) is equal to

\[
\int_{-\infty}^{x_1} \int_{-\infty}^{x_2} \int_{-\infty}^{x_3} \cdots \int_{-\infty}^{x_n} f_{x_1; x_2; x_3; \ldots, x_n}(x_1; x_2; x_3; \ldots; x_n) \, dx_1 \, dx_2 \cdots dx_n
\]

For purposes of the structural analysis, the random variables \( x_1, x_2, x_3, \ldots, x_n \) (i.e., the various sets of fractures) have been assumed to be mutually independent. In other words, at the geological time of fracture formation, the intensity with which fractures of a set developed was neither dependent upon nor influenced by the presence or absence of any of the other sets within the rock mass. This implies that for all values of \( x_1; x_2; \ldots; x_n \)

\[
f_{x_1; x_2; x_3; \ldots, x_n}(x_1; x_2; x_3; \ldots; x_n) = f_{x_1}(x_1) \cdot f_{x_2}(x_2) \cdots f_{x_n}(x_n)
\]
The average intensity of all fracture sets combined may be determined by the sum of the independent normal variates \( x_1, x_2, x_3, \ldots, x_n \), where each \( x_i \) has a \( n(\mu_i, \sigma_i^2) \) distribution.

Provided that the functions are not truncated by constraints, then according to Larson (1974), if \( Y \) represents the sum of the variates, i.e., \( Y \) is the total intensity of all fracture sets combined,

\[
Y = \sum_{i=1}^{n} x_i
\]

then, \( Y \) is a normal variate with mean and variance given by:

\[
\mu_Y = \sum_{i=1}^{n} \mu_i
\]

\[
\sigma_Y^2 = \sum_{i=1}^{n} \sigma_i^2
\]

The random variable \( Y \) may be considered as a \( n(\mu_Y, \sigma_Y^2) \)-variate. Thus, the total intensity of all fracture sets combined is a normal probability density function with a mean and variance equivalent to the sum of the means and variances for the various fracture sets, respectively.

It is evident from Figure 27 and earlier discussions that some of the functions describing fracture intensities are truncated by non-negativity and upper limit constraints. Therefore, it is not possible to combine these functions in the simple manner detailed above.
The problem was resolved by:

1. discretizing each continuous density function into a number of small, uniformly spaced, discrete class intervals, and

2. combining them by a series of cross-multiplications to yield a single probability density function for total fracture intensity. The task was expedited by means of a simple computer program.

The resulting function for variability between unit domains with respect to fracture set Nos. 1 - 10 combined (i.e., excluding bedding plane partings; set No. 11) is illustrated in Figure 28. The distribution is a three parameter lognormal probability density function, such that $\log_{10}(\text{Derived Intensity})$ plus a constant value of 6.0 is a normal variate.

Variability in the intensity of all fracture sets combined in hanging-walls of Orebody Nos. 5, 7 and 11 is summarized by a series of functions in Figures 29-31.
Cumulative Frequency Plot on Logarithmic Probability Paper for Local Variability in the Intensity of Fracture Set Nos. 1-10 Combined.

FIGURE 28.
Cumulative Frequency Plot on Probability Paper for Local Variability in the Intensity of Fracture Set Nos. 1-11 Combined within the Hanging-wall of Orebody No. 5 (i.e., Square Root of Intensity is a Normal-Variate).

FIGURE 29.
Cumulative Frequency Plot on Normal Probability Paper for Local Variability in the Intensity of Fracture Set Nos. 1-11 Combined Within the Hanging-wall of Orebody No. 11.

FIGURE 30.
Approximate Relationship Between Volume of Rock Mass & the Mean and Standard Deviation of Normal Probability Density Functions for Intensity of Fracture Set Nos. 1-10 and 1-11 (within Hanging-wall of Orebody No.5) Combined, Respectively (m²/m³).

FIGURE 31.
Approximate Relationship Between Volume of Rock Mass & the Mean and Standard Deviation of Normal Probability Density Functions for Intensity of Fracture Set Nos. 1-11 combined within the hanging-walls of Orebody Nos. 7 and 11, Respectively \( (m^2/m^3) \).

**FIGURE 31 (Cont.)**
The dolomitic shales are structurally complex.

Accordingly, the mechanical properties of the rock mass will be locally variable and the response of the mass under load will be complex. The objective of this chapter is to gain a broad, but statistically-based, appreciation of local variability in mechanical properties of the mine shales. This is achieved by representing the structural complexity in terms of a suite of material parameters.

Several interrelated factors will govern the response of complex models:

1. Range of mechanical properties and their frequency of occurrence within the mass.

2. Complexity and magnitude of the stress field around the opening(s).

3. Probability that the mechanical properties of the mass are exceeded along "critical paths" or potential failure zones within the mass.

6.1 Method of Analysis

As noted in Chapter 1, the mechanical properties employed in the present analysis are largely derived on the basis of published rock mass classification systems. The necessary geological input is provided by the statistical model for structural variability. The basic principles and
applicability of various classifications and other techniques are discussed in Baczynski (1979). A brief summary of the main results is included in Baczynski (1980f).

An overview of the task suggests that some aspects of the analysis may be readily undertaken by means of simple graphical transformations between the model for structure (or other field indices) and various rock mass parameters. However, the derivation of several other parameters via the more comprehensive classification systems is a slow, repetitive and very time-consuming process. The latter arises from the necessity to "sample" and evaluate a large number of possible structural situations before a statistical appreciation is gained of the range of parameters.

6.2 Computer Simulation

To expedite the evaluation process, a computer program GEOCLAS was written for the determination of several classification ratings, including the RMR-, Q-, RQD- and Modified RMR-indices as proposed by Bieniawski (1973, 1976), Barton et al. (1974), Deere et al. (1966) and Laubscher and Taylor (1976), respectively.

The principle employed in the computer program is quite simple. The above indices are determined for rock mass blocks with ground conditions simulated statistically on the basis of models for the intensity of each set of geological discontinuities, plus specified inputs for the other necessary parameters. The procedure is repeated several hundred times to permit a statistical model to be formulated for variability between blocks with selected dimensions (or volume). The minimum number of iterations employed for each block size in the present assessment is 1000. The capability of the program also extends to the determination of relationships between the various classification ratings.
A full FORTRAN listing of the program, description of input data requirements and examples of typical output are included in Baczynski (1979).

6.3 Model for RQD-Index

One of the parameters commonly included in rock mass classifications is the "rock quality designation" (RQD)-index proposed by Deere et al. (1966). The index is modified core recovery percentage in which all the pieces of sound core exceeding 0.1 metre in length are counted as recovery.

To permit this parameter to be manipulated in parallel with the statistical model for geological structure, it was necessary to establish a correlation basis between fracture frequency and RQD-index rating.

A literature survey on the topic indicates marked differences between various sources (Baczynski 1979, 1980f). The mean trends adapted from Deere et al. (1966) for a metamorphic rock type, and proposed by Priest and Hudson (1976) for sedimentary rocks and by Barton et al. (1975) for essentially igneous rocks, as well as by Kulhawy (1978) from theoretical considerations, are illustrated in Figure 32. Further discussion on the subject is presented in Hudson and Priest (1979) and Goodman and Smith (1980). It is evident from the results that no single method can be considered to have universal application.

The mean correlation established at the mine on the basis of diamond drill core data is also illustrated in Figure 32. The scatter of individual RQD values about the mean trend is about ±15 per cent.

The results are in extremely good accord with those of Priest and Hudson (1976), especially for fracture frequencies less than 20 per metre. The confidence of the trend at higher fracture frequencies is reduced by the
Relationship Between Fracture Frequency and RQD-Index: Mount Isa Mine

FIGURE 32.
lack of appropriate data.

It is interesting to note that both the Mount Isa Mine and the Priest and Hudson trends have been derived for sedimentary strata. These results suggest that one of the governing factors in the observed relationships may be rock type.

6.4 Basis of Model for Rock Mass Strength

6.4.1 Intact Rock Mass

The mean unconfined compressive and tensile strengths of intact rock core samples are 185 MPa and 20 MPa, respectively. A more detailed discussion of core strengths is presented in Chapter 2 of this thesis. Considerable variability is apparent in the results, even for texturally similar test samples. The variation appears to be further compounded by the anisotropic effect of sedimentary layering within the dolomitic shales.

It is anticipated that a significant proportion of the observed variability is a function of test sample dimensions. Therefore, it is unlikely that the model derived for relatively small test samples can be extended directly to describe the variation in the strength between larger rock mass blocks.

A simple example will readily demonstrate the effect of sample size on variability. Assuming that the core strengths are statistically independent and that the range of strength values for 54 mm diameter cores may be
described by a normal probability density function with a mean \( \mu \) and variance \( \sigma^2 \), then the corresponding model for variability between unit volumes (1.0 m\(^3\)) of the mass may also be expressed in terms of a normal function, but with a mean \( \mu' \) (i.e., \( \mu \), suitably adjusted by some scale factor to account for strength reduction due to intergranular defects) and a variance of about \( 0.002 \times \sigma^2 \).

It is apparent that the variability between unit volumes of intact rock mass is very small in comparison to that between the test samples. This suggests that any potential variability in the strength of intact rock mass blocks is likely to be imposed only by the sedimentary layering of the shales.

6.4.2 Discontinuous Rock Mass

So far, the discussion has been limited to the strength of intact rock blocks, whereas the dolomitic shales are characterized by the presence of geological discontinuities.

Published results of field and laboratory testing, as well as physical modelling studies of blocky systems, clearly demonstrate that the strength of rock masses is appreciably reduced by the presence of geological weakness planes. In instances of highly fractured rock, there is little relationship between the strength of the intact rock and that of the discontinuous mass.

Most experimental information is available for coal and, therefore, does not have direct application to the hard rock environment at the Mount Isa Mine. A comprehensive discussion of the subject is presented in Lama and Vutukuri (1978). A number of the techniques reviewed in the present study are included in Baczynski (1979).
The dolomitic shales are characterized by the presence of several sets of geological discontinuities as well as by local variability in their intensity per unit volume of the mass. Therefore, some degree of local variability may be anticipated in the strength of the rock mass.

A number of field and laboratory studies that are reported in literature have attempted to derive a parallel model between intensity of fracturing and rock mass strength. None of the suggested methods are entirely satisfactory and it is also difficult to extend them to derivation of a statistical model for variability. However, the work of Hansagi (1965, 1974), and possibly that of Stimpson and Ross-Brown (1979), appear to be the most promising with respect to the present analysis.

Hansagi proposed a diamond drill core classification index (essentially based on the intensity of fracturing) which, when coupled with the unconfined compressive and tensile strengths of the cores, permits estimates of rock mass strength. The index has been developed for a rock mass with broadly similar geological characteristics, apart from sedimentary bedding, to those of the mine shales (i.e., tensile strength of intact rock cores, rock density, and similar ranges of fracture intensities as suggested by the values determined by the C-factor index). Therefore, the method offers a potentially useful approach and accordingly it has been adopted in the present study.

The relationship between C-factor index rating and the total intensity of fracturing within the mine shales is presented in Table 3. Non-tabulated values may be approximated by linear interpolation.

Estimates of effective rock mass friction angle and cohesion are based on Bieniawski (1973, 1976). Figure 1 of the earlier technical paper presents a relationship between the unconfined compressive strength of intact rock cores, spacing of joints and the cohesion and friction angle of a rock mass. Section D of Table II in the latter publication suggests ranges of friction angles and cohesion values for rock masses in each of the five RMR-index ratings.

The proposed cohesion values are rather low. They appear to be characteristic of completely blocky rock masses. The discontinuous nature
of most weakness planes within the dolomitic shales at the mine suggests that the mass will only be completely blocky locally. Therefore, it is anticipated that the rock mass as a whole will possess a greater degree of cohesion than may be predicted on the basis of Bieniawski (1973, 1976).

In the present analysis, cohesion has been deduced from a Mohr-circle representation of the unconfined rock mass strength and a simple, linear failure envelope incident upon the stress circle. The reduction factor proposed by Hansagi (1965, 1974) is used to estimate the unconfined compressive strength of the rock mass. Friction angles are based on the values suggested by Bieniawski (1976). The procedure employed to determine cohesion is illustrated graphically in Figure 32A.

It is appreciated that the approach is very simplistic and possibly, a little unrealistic in some aspects. A linear failure envelope is assumed. Therefore, some cohesion will be predicted in all instances where the unconfined compressive strength of the mass exceeds zero. It is also apparent that the magnitude of the derived estimate will not only be governed by the unconfined compressive strength, but will also be a function of the friction angle. For example, rock masses with identical unconfined compressive strengths but different friction angles would automatically possess different cohesions.

The true shape of the failure envelope can only be defined by an extensive field testing programme. In the absence of such tests, some assumptions have to be made. A linear failure envelope is considered to present a reasonable first-order approximation.
\[ \sigma_1 = \text{Unconfined compressive strength of rock mass.} \]

\[ \sigma_3 = 0.0 \text{ (Confining stress).} \]

\[ \sigma_n = \text{Normal Stress.} \]

\[ \tau = \text{Shear Stress.} \]

\[ \phi = \text{Rock Mass Friction Angle.} \]

\[ c = \text{Rock Mass Cohesion.} \]

Derivation of Rock Mass Cohesion.

**FIGURE 32A.**
TABLE 3.

Relationship Between C-Factor and Intensity of Fracturing per Metre.

<table>
<thead>
<tr>
<th>Intensity / Metre</th>
<th>C-Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>1.00</td>
</tr>
<tr>
<td>3.0</td>
<td>0.79</td>
</tr>
<tr>
<td>5.5</td>
<td>0.65</td>
</tr>
<tr>
<td>7.0</td>
<td>0.56</td>
</tr>
<tr>
<td>8.5</td>
<td>0.50</td>
</tr>
<tr>
<td>9.5</td>
<td>0.45</td>
</tr>
<tr>
<td>24.0</td>
<td>0.13</td>
</tr>
<tr>
<td>56.0</td>
<td>0.00</td>
</tr>
</tbody>
</table>

6.5 Basis of Model for Rock Mass Modulus

The rock mass is a multi-phase system composed of intact rock and geological discontinuities. Elastic properties of intact rock are essentially dependent upon the mineralogy of the rock, whereas the elastic properties of rock discontinuities depend on their length, tightness, waviness, degree of interlock and matching between asperities, the elastic properties of the infilling materials and, commonly, the magnitude of the confining stress field.
6.5.1 Modulus of Intact Rock

The effective Young's modulus of intact rock may be determined by the methods described in Lama and Vutukuri (1978). One possible approach is indicated below:

$$\frac{1}{E_{\text{eff}}} = \frac{V_a}{E_a} + \frac{V_b}{E_b} + \frac{V_c}{E_c} + \ldots + \frac{V_n}{E_n}$$

where, \(V_a, V_b, V_c, \ldots, V_n\) are the percentages of the minerals in the rock as determined from modal analysis and \(E_a, E_b, E_c, \ldots, E_n\) are the respective Young's modulus values.

An alternative approach is:

$$E_{\text{eff}} = V_a E_a + V_b E_b + V_c E_c + \ldots + V_n E_n$$

where,

1. if the shape of the grains is spherical, the volumetric (\(V_a, V_b, \text{etc.}\)) percentage of the minerals is taken into consideration, or

2. if the shape is not spherical and the particles have a preferred orientation, the percentage surface area of the minerals along a section parallel to the direction of application of the stress is used in calculations. This allows a determination of modulus of anisotropy in different directions.

According to Lama and Vutukuri (1978), the first equation gives the upper limit whereas the latter gives the lower limit. The actual value is located somewhere in between the two limits.

As noted in Chapter 2 of the thesis, the mean modulus parallel and normal to sedimentary layering of the shales is 104 GPa and 91 GPa, respectively.
The overall mean value is about 100 GPa.

6.5.2 Theoretical Approach to Rock Mass Modulus

The elastic properties in any direction through a rock mass may be determined from a consideration of the elastic properties of the intact rock material in that direction as well as the orientation, spacing and the normal and shear stiffness of the geological discontinuities within the mass. For example, in the case of an orthogonally jointed rock mass, the modulus in the $i$-th direction may be determined by means of relationships presented in Lama and Vutukuri (1978) and Kulhawy (1978):

$$E_i = \frac{1}{\frac{1}{E_r} + \frac{1}{S_i K_{ni}}}$$

where,

- $E_i$ = elastic modulus of the rock mass in the $i$-th direction,
- $E_r$ = elastic modulus of the intact rock material,
- $S_i$ = spacing of discontinuities in the $i$-th direction,
- $K_{ni}$ = normal stiffness of the discontinuities in the $i$-th direction.

Similarly, the shear modulus in the $ij$-th plane may be determined by:

$$G_{ij} = \frac{1}{\frac{1}{G_r} + \frac{1}{S_i K_{si}} + \frac{1}{S_j K_{sj}}}$$

where,
Some estimates of the elastic properties of intact rock, geological discontinuities and rock masses have been reported in Goodman et al. (1968), Lama and Vutukuri (1978) and Kulhawy (1978). It is apparent from the available information that:

(1) Normal and shear stiffness of rock discontinuities may assume a wide range of values.

(2) Measured stiffness is likely to be dependent upon the magnitude of the confining stress field.

(3) Estimates of rock mass modulus are very sensitive to the stiffness values assumed for the discontinuities.

Therefore, the theoretical approach to the determination of rock mass modulus is only useful if supported by reasonably accurate field tests undertaken within the rock mass that is being evaluated. In the absence of such tests, the reliability of estimates derived by the theoretical approach must be at least as suspect as those determined on the basis of rock mass classifications. Accordingly, estimates determined on the basis of classifications may be viewed as providing adequate first order approximations of the parameter. The approximations are no less accurate than those
resulting from the theoretical approach where the stiffness of the rock discontinuities is not known but only estimated.

Several rock mass classification approaches are described in literature. Rock mass modulus may be estimated on the basis of either the relationship between the parameter and RQD-index as derived by Coon and Merritt (1970), Cording et al. (1971), Orr (1974) and others, or from the relationship with the RMR-index presented by Bieniawski (1975, 1978a, 1978b). Both sets of approaches yield similar mean and ranges of modulus values for the dolomitic shales (Baczynski, 1980f).

There are obvious limitations in each of the two index-approaches and neither of them can be anticipated to yield anything more than an approximate estimate. However, since the RQD-index is both easier to derive and to manipulate in parallel with a statistical model for variability in the intensity of fracturing, it has been employed in the present analysis.

The modulus reduction factor (i.e., $E_{\text{mass}} / E_{\text{intact rock}}$) for the dolomitic shales may be approximated by two linear equations (Baczynski, 1979):

$$\text{Modulus Reduction Factor} = 0.1 + 0.0014 \times \text{RQD}$$

for $\text{RQD} \leq 70$

$$= 0.0267 \times (\text{RQD} - 62.5)$$

for $\text{RQD} > 70$

The necessary input to the theoretical and the rock mass classification approaches is provided by the statistical models for local variability in the intensity of each fracture set and the total intensity of fracturing, respectively. Where the normal and shear stiffness of fractures or the relationship between total fracture intensity and RQD-index are known, the structural model may be sampled iteratively by the Monte Carlo process to derive a parallel model for variability in rock mass modulus.

It is apparent that the theoretical approach permits evaluation of directional variability, whereas estimates derived on the basis of the classification approach are limited to isotropic assumptions.
Although the conceptual aspects of determining the Poisson's ratio of composite systems such as soil and rock masses has been discussed by a number of authors, including Wardle and Gerrard (1972), Harrison and Gerrard (1972), Kulhawy (1978) and Lama and Vutukuri (1978), there appears to be little confirmatory field evidence to support the theoretical estimates. Most published information is restricted to intact rock cores. A typical example is that of Kulhawy (1975) who reports that the ranges for clastic and chemical sedimentary rock types (similar to the dolomitic shales) are 0.03 - 0.46 and 0.04 - 0.73, respectively.

Rock masses behave in a complex manner under load and various modes of local deformation, other than elastic, have been observed. As failure conditions are approached, deformations may include local slip along geological weakness planes and dilation caused by block rotation. These mechanism may operate locally at an early stage of loading, without necessarily resulting in complete collapse of the system. Therefore, it is doubtful whether the concept of elastic Poisson's ratio can be applied in a strict sense to describe the observed behaviour. Conceptually, it may be a little more appropriate to refer to it as the "lateral deformation ratio".

The dolomitic shales at the mine are structurally variable. Therefore, the lateral deformation ratio may also be expected to be locally variable, unless the stiffness of the discontinuities is identical to that of the intact rock or the discontinuities are present in the mass with equal intensities in each of the three coordinate directions.

Since the appropriate deformations have not been measured in the field, a somewhat subjective approach has been adopted for the determination of this parameter. It is intuitively anticipated that the ratio will increase (possibly logarithmically) with increasing intensity of fracturing. This view is also held by Dr. R.D. Lama of the CSIRO (personal communication) on the basis of his physical modelling studies. A range of about 0.2 - 0.5
has been suggested for the mine shales.

The general appropriateness of this range has been reaffirmed theoretically by application of the equations presented in Kulhawy (1978), where a normal stiffness of about 60 - 80 GPa is assumed for discontinuities, and fracture intensities are in accord with the statistical model for structure within the dolomitic shales. Higher ratios are probably unlikely under the confining loads that are effective at the Mount Isa Mine.

Laboratory evidence cited in Lama and Vutukuri (1978) indicates that Poisson's ratio increases with increasing axial load, possibly as a result of plastic deformation in test specimens, and that the ratio decreases with increasing lateral or confining load. However, the inclusion of such complex relationships is outside the scope of available numerical techniques and it is assumed in the present analysis that the ratio remains constant under different load conditions.

6.7 Statistical Model for Material Properties

6.7.1 Definition of Domains

Two types of structural domains are recognised within the dolomitic shales. They are the essentially barren shale "hanging-wall" and the economically mineralized "orezone". In the case of each orebody studied, the two domains are distinguished by different mean intensities of bedding plane partings (fracture set No.11). The mean intensity of each of the other fracture sets appears to be the same in both domains.
6.7.2 Material Types

EIGHTEEN (18) ISOTROPIC MATERIAL TYPES ARE EMPLOYED TO CHARACTERIZE MECHANICAL VARIABILITY WITHIN THE DOLOMITIC SHALES.

The range is intended to represent the full spectrum of local variability in ground conditions that are encountered at the mine. It extends from intact rock to highly fractured rock at the two ends of the scale. Each of the material types is defined in Table 4.

The derivation of "mean" and "+1 standard deviation" material property types for the hanging-wall and orezone domains of Orebody Nos. 5, 7 and 11 is based on corresponding set of structural conditions postulated for these areas. The relevant material types are indicated in Table 5. Since the ground conditions in Orebody Nos. 5 and 7 are very similar, only data for the former is presented in the table.

6.7.3 Frequency of Occurrence

The relative frequency with which each of the material types occurs within the rock mass parallels the model for local variability in geological structure.

The material model derived for unit volumes (1.0 m$^3$) within the hanging-wall and orezone domains of Orebody Nos. 5 and 11 is presented on probability paper in Figure 33.
<table>
<thead>
<tr>
<th>Material Type</th>
<th>Modulus (GPa)</th>
<th>Poisson's Ratio</th>
<th>Friction Angle (Degrees)</th>
<th>Cohesion (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Unconf. Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>95</td>
<td>0.20</td>
<td>45</td>
<td>37.0</td>
<td>8.4</td>
<td>178</td>
</tr>
<tr>
<td>2</td>
<td>80</td>
<td>0.22</td>
<td>44</td>
<td>34.0</td>
<td>7.7</td>
<td>165</td>
</tr>
<tr>
<td>3</td>
<td>71</td>
<td>0.24</td>
<td>43</td>
<td>31.0</td>
<td>6.6</td>
<td>146</td>
</tr>
<tr>
<td>4</td>
<td>63</td>
<td>0.26</td>
<td>42</td>
<td>27.0</td>
<td>5.5</td>
<td>124</td>
</tr>
<tr>
<td>5</td>
<td>45</td>
<td>0.28</td>
<td>41</td>
<td>22.0</td>
<td>4.6</td>
<td>104</td>
</tr>
<tr>
<td>6</td>
<td>37</td>
<td>0.30</td>
<td>40</td>
<td>19.0</td>
<td>4.0</td>
<td>87</td>
</tr>
<tr>
<td>7</td>
<td>35</td>
<td>0.32</td>
<td>40</td>
<td>18.0</td>
<td>3.6</td>
<td>78</td>
</tr>
<tr>
<td>8</td>
<td>33</td>
<td>0.33</td>
<td>39</td>
<td>16.0</td>
<td>3.2</td>
<td>69</td>
</tr>
<tr>
<td>9</td>
<td>31</td>
<td>0.34</td>
<td>39</td>
<td>14.0</td>
<td>2.8</td>
<td>59</td>
</tr>
<tr>
<td>10</td>
<td>29</td>
<td>0.35</td>
<td>38</td>
<td>12.5</td>
<td>2.4</td>
<td>52</td>
</tr>
<tr>
<td>11</td>
<td>27</td>
<td>0.36</td>
<td>37</td>
<td>10.8</td>
<td>2.0</td>
<td>44</td>
</tr>
<tr>
<td>12</td>
<td>25</td>
<td>0.38</td>
<td>36</td>
<td>8.4</td>
<td>1.2</td>
<td>33</td>
</tr>
<tr>
<td>13</td>
<td>23</td>
<td>0.40</td>
<td>35</td>
<td>5.8</td>
<td>1.0</td>
<td>22</td>
</tr>
</tbody>
</table>
### Table 4 (Continued)

**Rock Mass Material Properties**

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Modulus (GPa)</th>
<th>Poisson's Ratio</th>
<th>Friction Angle (Degrees)</th>
<th>Cohesion (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Unconf. Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>21</td>
<td>0.41</td>
<td>34</td>
<td>4.8</td>
<td>0.8</td>
<td>19</td>
</tr>
<tr>
<td>15</td>
<td>19</td>
<td>0.42</td>
<td>33</td>
<td>3.9</td>
<td>0.6</td>
<td>15</td>
</tr>
<tr>
<td>16</td>
<td>17</td>
<td>0.43</td>
<td>32</td>
<td>3.0</td>
<td>0.4</td>
<td>11</td>
</tr>
<tr>
<td>17</td>
<td>15</td>
<td>0.44</td>
<td>31</td>
<td>2.1</td>
<td>0.2</td>
<td>7</td>
</tr>
<tr>
<td>18</td>
<td>13</td>
<td>0.45</td>
<td>30</td>
<td>0.8</td>
<td>0.05</td>
<td>3</td>
</tr>
</tbody>
</table>

* Tensile strength deduced on basis of Hansagi (1965), but NOT used in VISPLAS analysis.
<table>
<thead>
<tr>
<th>Orebody</th>
<th>Domain</th>
<th>Material Property Types</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-1 Standard Deviation</td>
<td>Mean</td>
</tr>
<tr>
<td>5</td>
<td>Hanging-wall</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>Orezone</td>
<td>7</td>
</tr>
<tr>
<td>11</td>
<td>Hanging-wall</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>Orezone</td>
<td>8</td>
</tr>
</tbody>
</table>
Cumulative Frequency Distributions Plotted on Probability Paper for Occurrence of Material Types within Unit Volumes (1.0 m³) of the Rock Mass.

FIGURE 33.
Three different approaches are used in this chapter to assess the stability of unsupported stope hanging-walls and abutment areas:

(1) Rock Mass Classifications

(2) Two-dimensional, Continuum, Finite Element Method

(3) Probability of Failure Analysis.

THE PARAMETRIC STUDY IS LIMITED TO MODELLING OF THE SEQUENTIAL EXTRACTION OF A SINGLE, RECTANGULAR-SHAPED STOPE.

Stopes within Orebody Nos. 5 and 11 are considered. Results for Orebody No. 5 reflect the anticipated mining behaviour within Orebody No. 7. Similar geological conditions are indicated in both areas.

The objective of this chapter is to briefly introduce each of the three approaches used in the analysis and to summarize the main results. Where applicable, implications of the work are discussed.
7.1 Rock Mass Classifications

7.1.1 Classifications Considered

Four main classifications are reported in technical literature for assessing the stability of unsupported stope openings:

(1) RQD-Index: Deere et al. (1966)

(2) RMR-Index: Bieniawski (1973, 1976)

(3) Modified RMR-Index: Laubscher and Taylor (1976)

(4) Q-Index: Barton et al. (1974)

The RQD-index is the simplest parameter to derive of the four listed above. However, a literature review indicates considerable differences of opinion as to estimates of unsupported spans. The spans also appear to be extremely conservative in comparison to past experience at the mine. Therefore, the present analysis has been restricted to the latter three classifications.

Details of the three classifications and associated input data requirements are presented in Baczynski (1979).
7.1.2 Estimates of Stable Hanging-wall Spans

The range of stable hanging-wall spans is based on statistical input for geological structure.

In the present assessment, the hanging-wall is considered to be of unit thickness in the third dimension. This approach provides a basis for comparison between the results of rock mass classification estimates and those of 2-D finite element and 2-D probability of failure analyses. This assumption is necessary because structural variability between rock mass blocks is a direct function of block volume (see Figure 27 on pages 83-87). The greater the block volume, the smaller the variability between the mean structural conditions in the blocks. The various approaches employed in the analysis of hanging-wall stability assume a unit thickness in the third dimension.

The first two metres into the hanging-wall are considered to be of the greatest importance from the stability point of view. This arises because hanging-wall dilution in the silver-lead-zinc stopes at the mine is commonly associated with slabbing from this region of the wall. Depending on the thickness of the ore zone, even 2.0 metres of collapse may constitute significant dilution. In the narrow orebodies, tipified by the model adopted for Orebody No.11, such collapse may represent a dilution of 50-70 per cent.

In terms of the assessment undertaken by Bieniawski's RMR geomechanics classification, structural variability between stope hanging-walls is based on the statistical model for a rock mass block with a volume of 25.0 m$^3$. A 50.0 m$^3$ volume is the basis for analyses employing Laubscher and Taylor's (1976) and Barton et al.'s (1974) classifications.

Structural conditions in each model are generated by means of "Monte Carlo" sampling of the probability density functions for the intensity of each fracture set. Model for variability between the hanging-walls is in accordance with Figure 27 of this thesis. Pre-mining stresses, unconfined compressive and tensile strengths of intact rock cores, and
other input parameters are discussed in the body of the thesis and in Appendix I. The results of the analysis are summarized in Table 6 and in Figures 34-39. Each of the cumulative frequency distributions presented in the diagrams is based on a sample of 1000 analyses of structurally feasible hanging-walls. The results in Table 6 are presented in terms of the mean value and a range within two (2) standard deviations. The cumulative frequency distributions are plotted on probability paper.

Bieniawski (1973, 1976) designated upper and lower limits of application for span estimates. These were based on case histories. Spans smaller than the specified lower limit were stable irrespective of ground conditions. Those within the lower-to-upper limit range also represented stable spans. However, depending on the classification rating, they required some ground support. Spans outside the upper limit were considered unstable without ground support. Results in Figures 34-35 are presented in terms of the two defined limits, as well as an average trend. Estimates in Table 6 are based on the frequency distribution for the upper limit of application.

A relationship between cavability and the extent of undercut area (expressed in terms of "hydraulic radius") is presented in Table VI of Laubscher and Taylor (1976). The hydraulic radius is defined as the area of the undercut surface divided by the length of the perimeter around the undercut area. Thus, the hydraulic radius is not only a function of the extent of the undercut area, but also of its shape (geometry). It is further stated by the two authors on page 125 of their paper that ".... large open stopes can only be mined in competent ground and the stope should have hydraulic radius 20% less than required for caving a rock mass ....". In terms of the hanging-wall geometry in open stopes at the mine, the hydraulic radius ranges from about 0.5 times the span in narrow stopes to about 0.25 times the span for stopes where the hanging-wall is roughly equidimensional in its strike and down-dip extent. Conservatively, the hanging-wall span in the present analysis has been assumed as twice the hydraulic radius, with a resulting maximum permissible span of 60 metres. The results of the assessment on the basis of Laubscher and Taylor's classification are summarized in Figures 36-37.

One of the key parameters in the Q-index rating developed by Barton et al. (1974) is the "stress reduction factor" or SRF. Its value is largely
dependent on the unconfined compressive and tensile strengths of intact rock cores, in-situ stresses and mean rock mass blockiness. A number of frequency distributions for various values of SRF are presented in Figures 38 and 39. Estimates in Table 6 are based on SRF values of 0.75 and 2.0 for Orebody Nos. 5 and 11, respectively.

MARKEDLY DIFFERENT DIMENSIONS ARE SUGGESTED FOR SPANS OF UNSUPPORTED HANGING-WALLS BY EACH OF THE CLASSIFICATIONS.

These differences highlight the limitations that are inherent in the rock mass classification approach to stability analysis.

The mean spans determined for Orebody Nos. 5 and 11 range from 11 to 28 metres and 6 to 15 metres, respectively.
TABLE 6.

Hanging-Wall Spans (in Metres) on Basis of Rock Mass Classifications

<table>
<thead>
<tr>
<th>Orebody No.</th>
<th>Bieniawski Mean</th>
<th>95% Range</th>
<th>Laubscher &amp; Taylor Mean</th>
<th>95% Range</th>
<th>Barton et al. Mean</th>
<th>95% Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>11</td>
<td>7-16</td>
<td>25</td>
<td>17-33</td>
<td>28</td>
<td>16-48</td>
</tr>
<tr>
<td>11</td>
<td>6</td>
<td>3-15</td>
<td>15</td>
<td>7-32</td>
<td>11</td>
<td>4-26</td>
</tr>
</tbody>
</table>

FIGURE 34.

FIGURE 35.

FIGURE 36.

FIGURE 37.

FIGURE 38.

FIGURE 39.
7.2 Finite Element Modelling

7.2.1 Choice of Computer Program

A finite element program suitable for the envisaged two-dimensional analysis should exhibit the capability to:

(1) model elasto-plastic behaviour,
(2) allow several sets of material properties in each model,
(3) model sequential excavation of the opening.

Several generally available programs were considered to be potentially suitable for the analysis. A brief listing of the programs, their capabilities, and a discussion of program testing and difficulties encountered in the study are presented in Baczynski (1980b).

The program selected for the analysis is VISPLAS. This elasto-viscoplastic program was developed by Dr. J. Meek at the University of Queensland. Further details of the program are also presented in the above cited reference. In terms of the present study, it is sufficient to note that the continuum was discretized into eight(8)-node, isoparametric, quadrilateral elements. The advantages of employing isoparametric elements are discussed in Desai and Abel (1972) and in Desai and Christian (1977). Plane-strain conditions, the generalization of the Mohr-Coulomb yield criterion proposed by Drucker and Prager (1952), and an 'associated' plastic strain function are also assumed in the analysis. The viscous component is not involved.
Further details of the finite element method are available in a number of standard text-books, e.g., Zienkiewicz and Cheung (1968), Desai and Abel (1972), Gudehus (1977) and Desai and Christian (1977).

Capabilities of VISPLAS were extended by the writer to permit a greater number of material types to be used, to reduce output to selected elements and nodes and to present it in a more useful format (i.e., in terms of principal stress and displacement vectors) and to create files for subsequent computer plotting of the vectors. Source listing of modifications included in VISPLASNRB-version of the program, as well as input data requirements are presented in Baczynski (1980a).

All numerical analyses were undertaken on the UNIVAC 1100 computer system located at Mount Isa Mine.
7.2.2 Complementary Programs

Several minor complementary computer programs were written to facilitate data-file preparation, mesh modifications and mesh-plotting, as well as for plotting of stress and displacement vectors. To some extent, these programs are structured in a format that is only suitable for the present analysis.

A source listing of the programs and details of input data requirements are included in Baczynski (1980a).

7.2.3 Geometry of Model

The stability of sub-level open stope hanging-walls and abutments appears to be largely a function of the strike or horizontal span (length) of the hanging-walls. This arises because the dimensions of these stopes in the plane of bedding are considerably greater down-dip than in the horizontal direction. Arching occurs across the shorter distance and therefore the response of the wall is controlled by its horizontal dimension.

To study the effect of progressively extracting a stope and thereby increasing the hanging-wall span in the horizontal direction, the finite element mesh used in the analysis represents a sub-horizontal section through the stope. The section is a plane which strikes in a north-south direction and has a 25° easterly declination from the horizontal. This coincides with a plane normal to the sedimentary layering within the dolomitic shales. The modelled section is illustrated in Figure 40.
Figure 4.1

Cross-Section of Underground Sub-level Open Stopes Represented by Finite Element Model.
7.2.4 Mesh Formulation

There are several basic requirements which need to be examined and satisfied in any finite element analysis. These include:

(1) The continuum should be adequately discretized into elements, bearing in mind that finer subdivision is necessary in regions where stress concentrations are expected.

(2) The mesh represents an idealization of an infinite body by a finite model. Therefore, boundary conditions should be carefully considered to minimize their effect on the results of the analysis.

(3) A balance should be maintained between increased accuracy and the computational effort that is necessary to achieve it. A reduction in computational effort may be achieved in several ways, including reducing the number of elements in the mesh, minimizing bandwidth, reducing the number of iterations for acceptable convergence of solutions and reducing the number of excavation stages by which mining of a stope is simulated.

A discussion of the basic considerations and the overall problem is presented in Desai and Abel (1972).

One further consideration is imposed by the scope of the present analysis. To permit modelling and assessment of the effect on stability of heterogeneous rock mass properties, it is necessary to formulate a mesh with a sufficient number of elements in the vicinity of openings. Furthermore, since the variability within the rock mass is assumed to be a function of element size (i.e., larger elements display lower variability), the effectiveness with which local variability can be modelled is governed by the extent to which it is possible to discretize the hanging-wall and abutment areas in the immediate vicinity of the stope. A larger number of small elements permits the model to reflect in a more realistic manner the inherent variability that exists within the mass.
The base mesh and fixity of the boundary are illustrated in Figure 41. Several aspects of the mesh need to be noted:

(1) The mesh presumes a north-south axis of symmetry for the stope. This has the immediate benefit of considerably reducing the computational effort because the number of elements is halved. The use of this approach is justified on the basis that the prime objective of the analysis is to evaluate hanging-wall and abutment stability. Stability of these areas is important because failed ground is likely to be dislodged under the action of gravity, thus contributing to dilution of ore within the stopes. The scope of the study does not extend to footwalls. However, it should be appreciated that the assumed symmetry automatically implies identical or 'mirror image' responses in the footwall of the stope.

(2) There are two principal geological domains defined in the mesh, namely, the hanging-wall and orezone. Different statistical models for local variability in material properties are associated with each domain.

(3) The continuum in the vicinity of the opening has been discretized into a regular array of small, square elements. The detail present in this zone is justified in view of the model that is assumed for statistical variability in rock mass properties. The depth of this regular array into the hanging-wall and abutments is considered to be adequate for realistic modelling of potential failure zones around the stope. Since failed ground may be anticipated to dilate considerably (50 per cent is not an unrealistic estimate) after being dislodged, the maximum depth of potential collapse from a hanging-wall will be limited by the width of the stope. Accordingly, the depth of the regular array has been extended to roughly twice the width of the stope.
OUTER MESH

UL

Node with total fixity in x- and y-direction

Node with fixity in x-direction

Node with fixity in y-direction

FIGURE 41 - Base Mesh.
FIGURE 41 (Continued) - Base Mesh.
(4) The north-south and east-west distances from the edge of the opening to the boundary of the mesh are approximately 8 and 25 times the corresponding semi-diameter of the opening.

(5) Boundary conditions are illustrated in Figure 41. The four corner nodes defining the extremities of the mesh are completely constrained against movement (i.e., x- and y-displacements are zero) whereas all other nodes on the boundary of the mesh, except for those defining the excavation surface itself, are partially fixed. A residual stress field is assumed in the model before mining. To simulate excavation of the stope, equal stresses but acting in the opposite direction are applied along the surface of the excavation.

(6) The stope is excavated in five (5) sequential stages, with mining progressing from north to south. The small number of excavation stages chosen to simulate extraction of a stope may be somewhat unrealistic in the case of Orebody No.5 as it implies that the stope is mined in 15 metres deep cuts. However, the writer considers that the five stages are adequate for purposes of the analysis.

7.2.6 Mesh Types

Several versions of the base mesh described above were employed to model stopes in Orebody Nos. 5 and 11. Modifications to the dimensions or the deletion of elements from the base mesh were implemented by means of complementary programs described in Baczynski (1980a). Each mesh type is illustrated in Figure 42.
Dimensions of Various Mesh-Types Used in Analysis.

FIGURE 42.
Theoretically, the present plane-strain analysis belongs to a class of problems that involve continua for which all the boundaries are not defined. However, because an infinite body cannot be included in the assemblage, it has to be limited or made finite. This approach is feasible because the effect of the loading decreases with increasing distance from the point of its application. Moreover, it is possible to determine the significance and the extent of error which arises from the finite boundaries.

The adequacy of the base mesh, especially the effect of the fixed finite element boundaries on displacements in the vicinity of the opening, was tested on the CSIRO CYBER 7600 system by means of the boundary integral program BITEM developed by Crotty and Wardle (1978). Infinite boundaries are assumed in this analysis. The comparison was undertaken on the basis of Mesh-type 1 and mean material properties for the hanging-wall and orezone domains of Orebody No.5.

The results of mid-span hanging-wall displacements for various stages of excavation are illustrated in Figure 43. Both sets of results are in good accord, especially within a 40-50 metres wide zone around the opening. This zone coincides with areas of more immediate interest in terms of the present analysis.
Mid-span Displacement of Hanging-wall at Various Excavation Stages: Comparison between Boundary Integral and Finite Element Programs.

FIGURE 43.
7.2.8 Stress Field Assumed in Analysis

Rock load conditions have been derived on the basis of past stress measurements undertaken at the mine (as reported in Baczynski, 1977c) as well as discussions with mine personnel during late 1977 (Mr. M.F. Lee, personal communication).

The stress field assumed for the present analysis is indicated in Table 7. It corresponds with residual rock load conditions at 13-15 levels of the mine. The relative magnitude of these values has been subsequently reaffirmed in Bridges (1979b).

However, to test the sensitivity of the analysis to different pre-mining stress fields, several additional computer analyses were also undertaken. These runs were considered desirable to assess the response (i.e., displacements and stresses around the stope) of the rock mass under the range of different load conditions that are likely to occur at the mine. Assumed pre-mining stresses are indicated in Table 8.

7.2.9 Extent of Finite Element Modelling Programme

Modelling programme was comprised of 51 finite element computer runs. Brief summary of the work is indicated in Table 9. A comprehensive discussion of each model, including an indication of the spatial distribution of various material types within the mesh, is presented in Baczynski (1980b).
TABLE 7.

Magnitude of Pre-Mining Stress Field Assumed in Modelling

<table>
<thead>
<tr>
<th>Stress Type</th>
<th>Magnitude (MPa)</th>
<th>Field</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Principal - $\sigma_1$</td>
<td>36</td>
<td>090/30* (normal to bedding)</td>
<td>$\sigma_y$</td>
</tr>
<tr>
<td>Intermediate - $\sigma_2$</td>
<td>26</td>
<td>270/60 (down-dip of bedding)</td>
<td>$\sigma_z$</td>
</tr>
<tr>
<td>Minor - $\sigma_3$</td>
<td>16</td>
<td>000/00 (along strike of bedding)</td>
<td>$\sigma_x$</td>
</tr>
</tbody>
</table>

* Expressed as dip direction / dip angle from horizontal.

TABLE 8.

Sensitivity Testing: Different Initial Stresses

<table>
<thead>
<tr>
<th>Load Type No.</th>
<th>Magnitude of Stress (MPa)</th>
<th>$\sigma_y$</th>
<th>$\sigma_z$</th>
<th>$\sigma_x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1**</td>
<td>36</td>
<td>26</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>26</td>
<td>26</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>16</td>
<td>26</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>12</td>
<td>26</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>16</td>
<td>26</td>
<td></td>
</tr>
</tbody>
</table>

** Stress field assumed for mine at 13-15 levels.
TABLE 9.

Finite Element Modelling Programme Using VISPLAS.

<table>
<thead>
<tr>
<th>Orebody No.</th>
<th>Mesh Type*</th>
<th>Material Types#</th>
<th>No. of Runs</th>
<th>Analysis Type</th>
<th>Maximum No. of Iterations</th>
<th>Accuracy Achieved (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1</td>
<td>Mean</td>
<td>5</td>
<td>Elastic: Sensitivity Testing</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>1</td>
<td>Elasto-plastic: Sequential mining</td>
<td>15</td>
<td>3-5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-1 Standard Deviation</td>
<td>1</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>+1 Standard Deviation</td>
<td>1</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Statistically Heterogeneous</td>
<td>16</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>11</td>
<td>2</td>
<td>Mean</td>
<td>1</td>
<td>Elasto-plastic: Sequential Mining</td>
<td>15</td>
<td>10-15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-1 Standard Deviation</td>
<td>1</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>+1 Standard Deviation</td>
<td>1</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>1</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-1 Standard Deviation</td>
<td>1</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>+1 Standard Deviation</td>
<td>1</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Statistically Heterogeneous</td>
<td>9</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&quot;</td>
<td>3</td>
<td>&quot;</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>Modified</td>
<td>3</td>
<td>&quot;</td>
<td>5</td>
<td>Elastic: Single-stage Mining</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td>Modified</td>
<td>4</td>
<td>&quot;</td>
<td>4</td>
<td>Elasto-plastic: Single-stage mining</td>
<td>15</td>
<td>2-16</td>
</tr>
</tbody>
</table>

* Figure 42 indicates Mesh types.  
# Material type parameters are defined in Tables 4 and 5.
7.3 Results of Finite Element Modelling

The extent of plastic yielding, zones of tensile stress, and principal stress and displacement vectors for the five stages of excavation are presented in Baczynski (1980b, 1980c) for each finite element run undertaken. The present discussion summarizes the main trends.

To facilitate the description of the results, areas of the finite element mesh in the immediate vicinity of the stope have been designated as "hanging-wall" and "abutment" domains. The extent of these from the surface of the opening into the rock mass is illustrated in Figure 44.

7.3.1 Extent of Plastic Yield

Results of the analysis are summarized in Tables 10 - 13 and in Figures 45 - 48.

Figures 45 and 46 present frequency distributions for the number of elements yielding plastically within the hanging-wall domain of stopes. Results are for each stage of excavation. A star-symbol is used to designate models with homogeneous material properties. The code "M", "+1" and "-1" denotes models with "mean", "+1 standard deviation" and "-1 standard deviation" material properties within hanging-wall and orezone domains. Large dots are used to indicate results for heterogeneous models with statistically variable material properties.

Figures 47 and 48 summarize the relative frequency with which particular elements around the stope yield plastically. A legend
2-D FINITE ELEMENT MODELLING OF SHALE HANGING-WALLS: LEAD-ZINC OREBODIES: MOUNT ISA MINE, QUEENSLAND

KEY DIAGRAM INDICATING ELEMENTS THAT ARE CONSIDERED TO BE LOCATED WITHIN HANGING-WALL(E) AND ABUTMENT(%) DOMAINS.

FIGURE 44.
TABLE 10 - Stability of 5 Orebody Stope with Uniform (Homogeneous) Material Properties

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Hanging-wall</th>
<th>Abutments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean:</td>
<td>40m strike span stable, but local failure is initiated at both edges of the wall when its strike span is extended to 55m. Depth of failed ground is 5m.</td>
<td>Stable when hanging-wall span is 25m, but extensive failure is suggested when the span is extended to 40m.</td>
</tr>
<tr>
<td>+1 Standard Deviation:</td>
<td>55m strike span is stable, but local failure is initiated at one edge of the wall when its span is extended to 70m. Depth of failed ground is 5m.</td>
<td>As above.</td>
</tr>
<tr>
<td>-1 Standard Deviation:</td>
<td>25m strike span is stable, but failure is initiated at both edges of the wall when its span is extended to 40m. Depth of failed ground is 10m.</td>
<td>As above.</td>
</tr>
<tr>
<td>Strike Span of Hanging-wall (m)</td>
<td>Suggested Probability of Failure on Basis of 16 Runs</td>
<td></td>
</tr>
<tr>
<td>--------------------------------</td>
<td>----------------------------------------------------</td>
<td>-----------------</td>
</tr>
<tr>
<td></td>
<td>Local Failure</td>
<td>Extensive Failure</td>
</tr>
<tr>
<td>10</td>
<td>±5%</td>
<td>-</td>
</tr>
<tr>
<td>25</td>
<td>±30%</td>
<td>-</td>
</tr>
<tr>
<td>40</td>
<td>±80%</td>
<td>5-10%</td>
</tr>
<tr>
<td>55</td>
<td>100%</td>
<td>40-50%</td>
</tr>
<tr>
<td>70</td>
<td>100%</td>
<td>±95%</td>
</tr>
</tbody>
</table>
### TABLE 12 - Stability of 11 Orebody Stope with Uniform (Homogeneous) Material Properties

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Hanging-wall</th>
<th>Abutments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean:</td>
<td>A 2m strike span is stable, but local failure extending ±1m into the wall is initiated at both edges of the wall when its strike span is extended to 5m. Extensive failure of 2-4m into the wall is apparent when the span of the wall increased from 8 to 14m.</td>
<td>Stable when hanging-wall strike span is 2m. Local failure occurs when the span is extended to 5m and becomes extensive when the span is increased to 8m.</td>
</tr>
<tr>
<td>+1 Standard Deviation:</td>
<td>A 5m strike span is stable, but local failure extending ±1m into the wall is initiated at both edges of the wall when its span is extended to 8m. Failure appears to be confined to the edges of the wall as its strike span is extended to 14m.</td>
<td>Stable when hanging-wall strike span is 5m, but extensive failure is suggested when the span is extended to 8m.</td>
</tr>
<tr>
<td>-1 Standard Deviation:</td>
<td>The minimum 2m span that was modelled is unstable.</td>
<td>Stable when hanging-wall strike span is 2m, but extensive failure is apparent at greater spans.</td>
</tr>
</tbody>
</table>
### TABLE 13 - Stability of 11 Orebody Stopes with Heterogeneous (Statistically Variable) Material Properties

<table>
<thead>
<tr>
<th>Strike Span of Hanging-wall (m)</th>
<th>Suggested Probability of Failure on Basis of 9 Runs</th>
<th>Abutments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Local Failure</td>
<td>Extensive Failure</td>
</tr>
<tr>
<td>2</td>
<td>±70%</td>
<td>±10%</td>
</tr>
<tr>
<td>5</td>
<td>±99%</td>
<td>±70%</td>
</tr>
<tr>
<td>8</td>
<td>100%</td>
<td>±99%</td>
</tr>
<tr>
<td>11</td>
<td>N/A</td>
<td>100%</td>
</tr>
<tr>
<td>14</td>
<td>N/A</td>
<td>100%</td>
</tr>
</tbody>
</table>
Orebody No. 5: Cumulative Frequency Distributions (plotted on Probability Paper) for Number of Yielding Elements in Hangingwall of Stope with Uniform and Heterogeneous Material Properties.

FIGURE 45.
 Orebody No.11: Cumulative Frequency Distributions (plotted on Probability Paper) for Number of Yielding Elements in Hanging-wall of Stope with Uniform and Heterogeneous Material Properties.

**FIGURE 46.**

+1, M, -1 = +1 Standard Deviation, Mean, -1 Standard Deviation Uniform Material Properties.

○ = Statistically Heterogeneous Properties.
Excavation Stage No.1

Excavation Stage No.2

Excavation Stage No.3

Excavation Stage No.4

Excavation Stage No.5

* * * corresponds to yielding elements in 1-24%, 25-49%, 50-74% and 75-100% of finite element runs, respectively.

OREBODY: EXTENT OF PLASTIC YIELDING AROUND STOPE WITH STATISTICALLY VARIABLE MATERIAL PROPERTIES.

FIGURE 47.
Excavation Stage No. 1

Excavation Stage No. 2

Excavation Stage No. 3

Excavation Stage No. 4

Excavation Stage No. 5

• • • • corresponds to yielding elements in 1-24%, 25-49%, 50-74% and 75-100% of finite element runs, respectively.

11 OREBODY: EXTENT OF PLASTIC YIELDING AROUND STOPE WITH STATISTICALLY VARIABLE MATERIAL PROPERTIES.

FIGURE 48.
accompanies each diagram. The intensity of shading is proportional to the observed yield rate.

Modelling of stopes in Orebody No. 5 suggests that:

(1) hanging-walls should remain stable for strike spans not exceeding 25 metres, but some local failures of the wall and abutment areas is anticipated,

(2) considerable instability of hanging-wall and abutment areas is envisaged at strike spans of 40 metres or greater.

The results for Orebody No. 11 suggest that:

(1) there is a likelihood of hanging-wall collapse and abutment failure even in stopes with a hanging-wall strike span of 2 metres.

(2) extensive failure is likely around stopes with larger strike spans.

It is evident from the analysis that:

MODELS WITH STATISTICALLY HETEROGENEOUS MATERIAL PROPERTIES CONSTITUTE A MORE REALISTIC APPROACH TO STABILITY ANALYSES.

Such models are extremely useful in demonstrating the potential extent of ground failure around openings and the excavation stage at which failure is initiated. A typical example of the contrast between models with homogeneous and statistically heterogeneous material properties is illustrated in Figures 49 and 50.

These plots illustrate the extent of plastic yielding. Various codes are used to designate incidence of yield, failure mode (i.e., tensile or compressive) and zones of tensile stresses. An appropriate
Extent of Plastic Yield in Model with Mean, Uniform (Homogeneous) Material Properties:
Orebody No. 11

FIGURE 49.
LEGEND

1 = Element yielded in compression during present stage of excavation.
1* = Element yielded in tension during present stage of excavation.
2 = Element yielded during an earlier stage of the excavation sequence.
2* = Element yielded during an earlier stage of the excavation sequence and is currently in tension.
4 = Element has not yielded but is currently in tension.

Typical Example of the Extent of Plastic Yield in Model with Statistically Heterogeneous Material Properties: Orebody No.11

FIGURE 50.
Rigorous statistical assessment of the finite element results is limited by the small number of runs undertaken with respect to each orebody. However, on basis of available evidence, the response of the two model types is significantly different at the 95% level of confidence.

MODELS WITH STATISTICALLY HETEROGENEOUS MATERIAL PROPERTIES PREDICT:

(1) GREATER EXTENT OF PLASTIC YIELD

(2) FAILURE MAY BE INITIATED AT AN EARLY STAGE OF EXCAVATION.

Distribution functions for the number of yielding elements in the two model types are presented in Figures 45 and 46.

7.3.2 Displacements

The mid-span hanging-wall displacements are illustrated graphically in Figures 51 and 52. The five stages of excavation within the two orebodies are represented.

The curves indicate the mean displacement of the walls. They are derived on the basis of models with statistically heterogeneous material properties. Variability between models is expressed in terms of the standard deviation which is stated as a percentage of the plotted mean value. For example, a mean value of 60 mm and a standard deviation of 4 per cent means that the range of displacements observed at that location within the hanging-wall may be
OREBODY - Statistical Material Properties

Legend:.... 10m = Mean Displacement curve for hanging-wall span of 10 metres;
(7%) = Standard deviation of normal distribution expressed as a percentage of Mean Value.
II OREBODY - Statistical Material Properties

Legend:...... Sm = Mean displacement curve for hanging-wall span of 5 metres;
................. (7%) = Standard deviation of normal distribution expressed as a percentage of Mean Value.

Distance into Hanging-Wall in Metres

Mid-span Hanging-wall Displacement in Millimetres
MID-ABUTMENT DISPLACEMENTS AT VARIOUS DEPTHS (0.0, 2.5, 5.0, 10.0, 15.0, 25.0, 35.0, 80.0, 130.0 METRES) INTO THE ABUTMENT OF A 20.0 METRES WIDE STOPE: 5 OREBODY - MEAN MATERIAL PROPERTIES.

(ON BASIS OF "BITEM" PROGRAM)

FIGURE 53.
represented by a normal probability density function with a mean of 60 mm and standard deviation of 2.4 mm.

Estimates of hanging-wall displacements for spans between those illustrated may be derived by interpolation between the curves.

The mid-abutment displacements for the five stages of excavation of a stope within Orebody No. 5 are presented graphically in Figure 53. Mean material properties have been assumed for the hanging-wall and orebody domains of the mesh. The curves represent the displacements at various distances into the abutments. The distance indicated adjacent to each curve corresponds to the distance (in metres) from the free face of the opening to the selected depth in the rock mass.

Examples of typical displacements at other locations around the model stope are summarized for each of the orebodies in terms of vector plots in Figures 54 and 55. The length and direction of vector corresponds to the magnitude and direction of displacement.

7.3.3 **Stresses**

Typical maximum and minimum principal stress vectors for each stage of excavation in models with homogeneous material properties are illustrated in Figures 54 and 55. Tensile stress vectors are represented by double lines.

Stresses range from about -10 MPa to 110 MPa. Magnitude depends upon location, stage of excavation and the spatial distribution of material types around the opening.
FIGURE 54 - Orebody No. 5: Model with Uniform Material Properties: Principal Stress and Displacement Vectors.
FIGURE 54 (Continued) - Orebody No.5: Model with Uniform Material Properties: Plot of Principal Stress and Displacement Vectors.
FIGURE 55 - Orebody No.11: Model with Uniform Material Properties: Plot of Principal Stress and Displacement Vectors.
FIGURE 55 (Continued) - Orebody No. 11: Model with Uniform Material Properties: Plot of Principal Stress and Displacement Vectors.
MAGNITUDE AND ORIENTATION OF TENSILE STRESSES WITHIN THE HANGING-WALL DOMAIN ARE MARKEDLY DIFFERENT IN MODELS WITH HOMOGENEOUS AND STATISTICALLY HETEROGENEOUS MATERIAL PROPERTIES.

Under the pre-mining stress field assumed in the analysis, tensile stresses act largely in a direction parallel to the strike of the hanging-wall (i.e., in the x-axis direction) for models with homogeneous material properties. In statistically heterogeneous models, a significant proportion of elements in the mesh are associated with relatively high tensile stresses acting near-normal to the strike of this wall (i.e., in y-axis direction). Examples of the observed variation are indicated in Figure 56.

In the case of the dolomitic shales at the mine, these stresses would contribute to separation of bedding planes within the walls of the stope. This result suggests a potential failure mechanism for hanging-walls. Unstable blocks of rock mass may be isolated from these walls by the combined process of failed abutments and separated bedding planes.

MODEL RESPONSE IS EXTREMELY SENSITIVE TO THE MAGNITUDE OF THE PRE-MINING STRESS FIELD.

Tensile stresses were not apparent within the hanging-wall of models tested under the stress conditions assumed in a sensitivity analysis; as described in Table 8. Hanging-wall and abutment displacements were also markedly different. The results demonstrate the need for the stress parameter to be defined as accurately as possible. Otherwise, it is difficult to validate the response of numerical models with field observations.
FIGURE 56 - Example of Typical Principal Stresses Observed in Finite Element Models with Uniform and Statistically Heterogeneous Material Properties.
7.4 Probability of Failure Analysis

7.4.1 Description of Technique

The rock mass is viewed as a heterogeneous continuum with statistically variable material properties. The technique assesses the probability of local ground collapse around openings. Basis of the analysis consists of:

(1) a stress field,

(2) a statistical model for local variability in the strength of the mass.

The following parameters are necessary for the analysis:

(1) Principal stresses at specified locations around the opening.

(2) Failure envelopes for each material type involved in the analysis.

(3) Probability of occurrence of each material type. This probability is specified for each recognised structural domain of the model, such as the hanging-wall and orezone.

Principal stresses may be determined by means of an elastic finite element or boundary element analysis. The latter two parameters are a function of sample size. In terms of the present analysis, the rock mass has been characterized by an assemblage of unit volumes. It is a system of domains, each with a volume of $1.0m^3$. Variability
in rock mass properties and the probability of occurrence of particular material types are dependent on domain dimensions. For example, large domains will display lower inherent variability between them. Therefore, a different model for probability of occurrence of various material types will apply in such instances.

The probability of local failure is estimated by a simple graphical procedure. The technique can be readily computerized to expedite the analysis.

The following steps are involved for each location that is evaluated within the continuum:

1. Construction of a Mohr-circle for the observed principal stresses. This circle is superimposed upon the plot of failure envelopes for the various material types.

2. Determination of material types that will yield under the prevailing stress field. This simply involves the identification of material failure envelopes that are cut by the Mohr-circle for stress.

3. Derivation of the cumulative probability of occurrence for all yielding material types. Probability may be determined either from a "Material Type" distribution function such as illustrated in Figure 33 for the appropriate structural domain, or from a look-up table listing the relative frequency of occurrence for each material type.


THE DESCRIBED METHOD PRESUMES THAT THE SPATIAL DISTRIBUTION OF VARIOUS MATERIAL TYPES WITHIN THE CONTINUUM IS RANDOM.
The procedure involved in the analysis of a hypothetical example is illustrated in Figure 57.

7.4.2 Assessment of Probability

A probability of failure analysis was undertaken for each excavation stage of stopes located within Orebody Nos. 5 and 11.

BITEM, the boundary element program developed by Crotty and Wardle (1978) was used to determine input stresses. The analysis was extended to openings with the alternate, pre-mining, stress fields indicated in Table 8.

A simple computer program was written to expedite the analysis. It processes the principal stress data from BITEM and outputs probabilities of failure for specified locations around the model opening.

Results are summarized in Figures 58 - 63. They are represented in terms of contours for the probability of local ground failure. The probability of failure within unit domains of the rock mass system is indicated.

The likelihood of more extensive failure can be readily deduced from these plots. The method employed is illustrated schematically in Figure 64. The boundary defining the potentially unstable zone is subdivided into "unit" distances and the probability of segment failure is determined. It should be noted that segment length is dependent upon the dimensions of domains used to characterize the rock mass system. In the present analysis, segment length is 1.0 metre.
LOOK-UP TABLE

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Probability of Occurrence ($P_i = %$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15%</td>
</tr>
<tr>
<td>2</td>
<td>40%</td>
</tr>
<tr>
<td>3</td>
<td>15%</td>
</tr>
<tr>
<td>4</td>
<td>15%</td>
</tr>
<tr>
<td>5</td>
<td>10%</td>
</tr>
<tr>
<td>6</td>
<td>5%</td>
</tr>
</tbody>
</table>

Probability of Failure = $P_3 + P_4 + P_5 + P_6 = 15\% + 15\% + 10\% + 5\% = 45\%$

FIGURE 57. - A Hypothetical Example of Probability of Failure Analysis.
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

5 Orebody: Stage 1 of Excavation Sequence with Hanging-wall Span of 10 metres.

Stress normal to hanging-wall = 36 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 25.7 mm = 10 metres.

FIGURE 58.
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

5 Orebody: Stage 2 of Excavation Sequence with Hanging-wall Span of 25 metres.

Stress normal to hanging-wall = 36 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 25.7 mm = 10 metres.

FIGURE 58 (Continued).
Contours of Probability for Occurrence of Local Ground Failure.

5 Orebody: Stage 3 of Excavation Sequence with Hanging-wall Span of 40 metres.

Stress normal to hanging-wall = 36 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 25.7 mm = 10 metres.

Figure 58 (Continued).
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

5 Orebody: Stage 4 of Excavation Sequence with Hanging-wall Span of 55 metres.

Stress normal to hanging-wall = 36 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 25.7 mm = 10 metres.

FIGURE 58 (Continued).
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

5 Orebody: Stage 5 of Excavation Sequence with Hanging-wall Span of 70 metres.
Stress normal to hanging-wall = 36 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 25.7 mm = 10 metres.

FIGURE 58 (Continued).
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

11 Orebody: Stage 1 of Excavation Sequence with Hanging-wall Span of 2 metres.

Stress normal to hanging-wall = 36 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 12.9 mm = 1 metre.

FIGURE 59.
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

II Orebody: Stage 2 of Excavation Sequence with Hanging-wall Span of 5 metres.

Stress normal to hanging-wall = 36 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 12.9 mm = 1 metre.

FIGURE 59 (Continued).
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

11 Orebody: Stage 3 of Excavation Sequence with Hanging-wall Span of 8 metres.

Stress normal to hanging-wall = 36 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 12.9 mm = 1 metre.

FIGURE 59 (Continued).
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

11 Orebody: Stage 4 of Excavation Sequence with Hanging-wall Span of 11 metres.

Stress normal to hanging-wall = 36 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 12.9 mm = 1 metre.

FIGURE 59 (Continued).
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

11 Orebody: Stage 5 of Excavation Sequence with Hanging-wall Span of 14 metres
Stress normal to hanging-wall = 36 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 12.9 mm = 1 metre.

FIGURE 59 (Continued).
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

5 Orebody: Stage 5 of Excavation Sequence with Hanging-wall Span of 70 Metres.

Stress normal to hanging-wall = 26 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 25.7 mm = 10 metres.

FIGURE 60
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

5 Orebody: Stage 5 of Excavation Sequence with Hanging-wall Span of 70 metres.
Stress normal to hanging-wall = 16 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 25.7mm = 10 metres.

FIGURE 61.
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

5 Orebody: Stage 5 of Excavation Sequence with Hanging-wall Span of 70 metres.

Stress normal to hanging-wall = 12 MPa; Stress parallel to hanging-wall = 16 MPa.
Scale: 25.7 mm = 10 metres.

FIGURE 62.
CONTOURS OF PROBABILITY FOR OCCURRENCE OF LOCAL GROUND FAILURE.

5 Orebody: Stage 5 of Excavation Sequence with Hanging-wall Span of 70 metres.

Stress normal to hanging-wall = 12 MPa; Stress parallel to hanging-wall = 26 MPa.
Scale: 25.7mm = 10 metres.

FIGURE 63.
Failure Probability for Zone = \( P_1 \times P_2 \times P_3 \times P_4 \times P_5 \)

\[ = 0.80 \times 0.70 \times 0.65 \times 0.60 \times 0.65 \]

\[ = 0.14 \text{ or } 14 \text{ per cent.} \]

**Figure 64** - Schematic Representation of Probability of Failure Analysis for Extensive Zones.
Since the model assumes a random spatial distribution for material types, the probability of failure for the delineated block is simply equal to the product of failure probabilities determined for the segments. For example, likelihood of zone instability that is delineated by five unit segments with associated failure probabilities of $p_1, p_2, p_3, p_4$ and $p_5$ may be determined by:

$$\text{Probability of Failure} = p_1 \cdot p_2 \cdot p_3 \cdot p_4 \cdot p_5$$

Where the probability of failure of each of "n" segments is the same, the likelihood of extensive instability is given by $p^n$.

7.4.3 Advantages and Limitations of Method

There are several advantages in this method of approach:

1. Technique is relatively simple.
2. Cost of analysis is negligible in comparison to the elasto-plastic finite element approach.
3. Only a single, two- or three-dimensional, elastic stress analysis is necessary for each particular test case.
4. Various alternate statistical models for local variability in material properties can be quickly evaluated. Analysis can be expedited by the use of simple computer programs.
5. There is no limit to the complexity of the model for rock mass strength (apart from properties that change with time or are displacement dependent such as strain...
hardening or softening). Directional properties may be easily included.

(6) Significance of alternate premining stress fields can be readily evaluated by means of a parametric study.

However, there are also three main disadvantages:

(1) Stress pattern around openings is based on an analysis of a homogeneous model. A single set of elastic parameters is assumed for each structural domain (e.g., hanging-wall or orebody) of the model. The present study has demonstrated that marked differences exist in stresses between models with homogeneous and statistically heterogeneous elastic properties. This aspect is completely ignored in the "probability of failure" analysis.

(2) The method is a static analysis. It does not consider load transfer, whereby the load previously sustained by a yielding element is redistributed to the remaining intact elements. It also fails to include gravity as a component of loading. In practical situations, this component becomes progressively more important as yielding increases.

(3) The method presumes that the spatial distribution of various material types is random within the continuum. This assumption is at variance with the "zonal" model developed for spatial distribution of fracture sets in rock masses. Complexity of the technique could be extended to include the "zonal" concept.

In spite of these limitations, the described technique is no less accurate than the slip-circle analysis employed in soil mechanics to assess slope stability. It serves to provide first-order estimates of the probability that local failures will occur around openings.
The approach may be extended to approximately simulate redistribution of excess loads. This would involve the undertaking of a series of related stress analysis runs. At each successive stage, the new boundary of the opening would coincide with some contour for the probability of failure (either for individual components within the system or for more extensive zones). The selected contour would correspond to some level of probability that the analyst considers to be unacceptable from the stability point of view.

Changes in the geometry of the opening will result in a new stress pattern. Assessment of this model may indicate further areas where the probability of failure is above the acceptable level of risk. The analysis is iterated until the extent of cumulative instability is assessed.
This chapter compares modelling predictions with mining experience in the silver-lead-zinc orebodies at Mount Isa.

The aim of this component of the project has been to assess past performance within sublevel open stopes and to derive a statistically based model for strike spans of stable hanging-walls. The incidence of local and extensive ground failures is also considered.

8.1 Practical Difficulties

A preliminary survey of records available at the mine suggested that it may be feasible to undertake the envisaged study. Apparent data-banks were provided by Mine Survey's, Mine Planning's, Mine Geology's and Mining Research Section's files. Discussions with long-term employees of the company also suggested a potential source of information.

Some aspects related to the proposed study were already assessed and documented, e.g., Brady (1975, 1977), Bridges (1979a, 1979b, 1980), Lang (1980), Lee and Bridges (1980).

However, closer examination of the task revealed certain practical difficulties:

(1) Volume of "potentially" useful records is considerable. These are sometimes in a form that is not readily accessible.
(2) Quality and detail of records is not consistent. Some discrepancies exist between various data sources. Personal bias is apparent in the description of ground failure severity. The relative importance of events is commonly proportional to stope dimensions (i.e., the larger the stope, the greater must be the volume of failed ground before it is considered as significant).

(3) Some records are not available and there are gaps in the information.

(4) Quality and reliability of geological data is variable. This is largely a reflection on the diminished importance that was attached to mapping of geological structures and core logging for rock mechanics purposes prior to the 1970's.

(5) The geology of some stope hanging-walls is more complex than assumed in the present study. Several rock types and structural environments are apparent along some walls. These include the presence of silica-dolomite, uniformly bedded shales, major folding, transverse faulting and strike shearing.

(6) Poor stoping practise, such as severe undercutting of hanging-walls, has contributed to some failures.

(7) Stability of some stopes was affected by other factors apart from geological structure, e.g., increased loads due to proximity of other mine workings.

(8) Unsupported hanging-walls were exposed by several different mining methods. Sublevel open stoping, long-hole benching, shrink-stopping and other methods have been used. Combinations of these techniques were employed in some stopes. The effect of each mining method on hanging-wall stability may be different.
(9) A large number of stope hanging-walls have similar dimensions. This arises because the orebodies are commonly extracted on a regular pattern of stope and pillars. Only limited information can be deduced in regard to the maximum dimensions for stable spans.

(10) Stopes have been mined at various depths below surface. Therefore, a common pre-mining stress field is not associated with all of them.

(11) Other factors are likely to modify the overall rock mass response. These include pillar dimensions, stope extraction sequence, blasting practice and stope-filling history. These contribute to some of the variability observed in hanging-wall stability.

8.2 **Scope of Analysis**

In view of the limitations suggested above, the depth of the study that was undertaken does not permit a rigorous back-analysis. However, some broad comparisons can be made between prototype and model response.

The investigation of Orebody Nos. 5 and 7 is restricted to stopes extracted between 7 and 15 levels at the mine. Stopes in Orebody No. 11 are located between 7 and 9 levels. These levels correspond to depths below surface of between 270 metres and 730 metres.

Considerable stoping was also undertaken above 7 level. However, records describing their behaviour are generally not available. Furthermore, the proximity of these stopes to the surface places
them in an environment that is markedly different to that which was modelled, i.e., lower stresses and greater degree of weathering of the rock mass.

8.3 Observed Response

A total of forty-five (45) stopes were examined in Orebody Nos. 5 and 7 combined, and seven (7) within Orebody No. 11.

Only 30 per cent of hanging-walls in the former two orebodies are located in uniformly bedded dolomitic shales. A further 20 per cent may be classed as a predominantly similar geological environment (i.e., less than 25 per cent of their surface area is associated with major fold zones, transverse faulting or the more massive "silica-dolomite" rock type). Therefore, about 50 per cent of the stopes have hanging-walls that are broadly similar to that modelled in the present analysis.

All stopes examined in Orebody No. 11 are located in uniformly bedded dolomitic shales or predominantly similar geological environment. However, it should also be noted that most are in close proximity to the contact with the silica-dolomite rock type.

The mean dimensions deduced for the horizontal or strike span of hanging-walls and for widths of stopes are summarized in Figures 65 and 66. These indicate that the excavated strike span of hanging-walls is generally greater than 25 metres. It is commonly about 30 - 40 metres.

Although not represented graphically, the down-dip extent of stopes is in the range of about 80 to 120 metres.
Orebody Nos. 5, 7 and 11: Relationship Between Strike Span of Hanging-walls and Width of Stopes.

FIGURE 65.
Orebody Nos. 5 and 7 Combined: Cumulative Frequency Distribution plotted on Probability Paper for the Strike Span of Hanging-walls.

FIGURE 66.
Geometry of individual stopes is rarely uniform across the opening. Variation is most pronounced in their width normal to bedding. Stope dimensions commonly reflect local changes in the thickness of the economic mineralization. Observed local variations are summarized in Table 14.

The stability of hanging-walls and abutments of stopes with various strike spans are indicated in Table 15. The results suggest that the walls and abutments are stable in stopes with hanging-wall strike spans not exceeding 30 metres. The probability of major collapse increases considerably as the span is extended to 40 metres.

Data for stopes with spans exceeding 40 metres is limited. The available results do not appear to be in accord with the trend deduced for stopes with smaller hanging-wall dimensions.

Some additional information is provided by the trial stoping experiment undertaken within Orebody No. 7 between 11 and 13 levels of the mine (Brady, 1975, 1977). The hanging-wall of the two stopes involved remained stable for spans of 31 metres and 39 metres, respectively. The pillar between the stopes deteriorated as its dimension in the north-south direction was progressively reduced to about 14 metres. Eventual failure of this pillar exposed a hanging-wall with an unsupported span of 80 metres. Considerable overbreak from this wall was recorded.

Hanging-wall and footwall displacements have been monitored in a number of stopes. Results of two case studies undertaken in parallel with the present research project are reported in Lang (1980) and in Bridges (1979a). Monitoring sites are located in Orebody No. 5 between 8 and 10 levels, and in Orebody No. 8 between 13 and 15 levels, respectively.

A mid-span hanging-wall displacement of about 25 mm is suggested in the Orebody No. 5 stope prior to failure of the wall. A mid-span footwall displacement of 10 mm is suggested at a strike span of 75 metres. Collars of instrumentation are located 35 metres and 45 metres from the respective walls. The observed displacements are
### TABLE 14.

**Case Study: Variations in the Width of Stopes (in Metres).**

<table>
<thead>
<tr>
<th>Orebody No.</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>23</td>
<td>8</td>
<td>42</td>
<td>4</td>
</tr>
<tr>
<td>7</td>
<td>12</td>
<td>7</td>
<td>27</td>
<td>3</td>
</tr>
<tr>
<td>11</td>
<td>8</td>
<td>4</td>
<td>17</td>
<td>2</td>
</tr>
</tbody>
</table>

### TABLE 15.

**Case Study: Stability of Hanging-walls & Abutments.**

<table>
<thead>
<tr>
<th>Hanging-wall Span (Metres)</th>
<th>Probability of Failure (%)</th>
<th>Orebody Nos. 5 and 7</th>
<th>Orebody No. 11</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Hanging-wall*</td>
<td>Abutment#</td>
</tr>
<tr>
<td>Less than 30</td>
<td></td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>30-35</td>
<td></td>
<td>9</td>
<td>25</td>
</tr>
<tr>
<td>36-40</td>
<td></td>
<td>33</td>
<td>50</td>
</tr>
<tr>
<td>Greater than 40</td>
<td></td>
<td>25?</td>
<td>0?</td>
</tr>
</tbody>
</table>

* In uniformly bedded dolomitic shales only.

# Irrespective of hanging-wall geology.
expressed relative to these positions. This indicates that the total displacement relative to an "infinite" boundary will be much greater.

In the Orebody No. 8 study, total displacements of about 25 - 35 mm and 50 - 60 mm were indicated for the hanging-wall and footwall, respectively, at a strike span of 80 metres for the stope. These values have been adjusted to remove the effect of instrumentation collar proximity to the opening. A comprehensive discussion of the pre-mining stress field, observed displacements and their interpretation is presented in Bridges (1979a). Total deformation is interpreted in terms of elastic and non-elastic components.

It should be noted that the hanging-wall of both groups of stopes is geologically complex. It is locally associated with the more massive silica-dolomite rock type and with major folding. The footwall in both instances is comprised entirely of shale. Significant hanging-wall collapses were encountered in both orebodies. Failures were "dish-shaped" and were dislodged from shale portions of walls. There was virtually no collapse from the silica-dolomite.

8.4 Rock Mass Classification Predictions

THERE ARE DIFFICULTIES IN DIRECT APPLICATION OF PUBLISHED ROCK MASS CLASSIFICATIONS TO ESTIMATES OF SPANS FOR UNSUPPORTED OPENINGS.

Estimates of stable spans for hanging-walls of sublevel open stopes in the silver-lead-zinc orebodies at the Mount Isa Mine appear to be conservative. This is particularly apparent with respect to the RMR-Geomechanics classification system proposed by Bieniawski (1973).
However, it must be appreciated that the RMR-system was basically designed for the evaluation of near surface structures. It was never intended to embrace mining situation. Its principal applications are to permanent engineering structures within rock masses that are commonly subjected to relatively low stresses. With respect to these, the classification provides an excellent system which is no more conservative than any of the others (Baczynski, 1980f).

The following three main factors confound the application of the RMR-index to mining situations:

1. The system does not provide for the incorporation of stress effects on stability.

2. Although the system attempts to determine stand-up time which could then be indirectly related to stability in mining situations, the proposed times are conservative with respect to past mining experience at the mine.

3. There is an upper limit of 20 metres for the maximum permissible span dimension. This value is extremely conservative.

The modified RMR-system proposed by Laubscher and Taylor (1976) and the Q-index developed by Barton et al. (1974) were proposed for, or extended to embrace, mining situations. Both classifications yield similar estimates for stable spans.

The determined values again appear to be conservative. However, the results are considerably better than those derived on the basis of Bieniawski's classification. The derived spans for Orebody Nos. 5 and 7 appear to be within 20 - 30 per cent of the average values suggested by past mining experience.

There is a marked discrepancy between field observations and stability estimates for stopes in Orebody No. 11. However, it should be appreciated that the structural model employed in the analysis
was formulated on the basis of investigations undertaken between 11 and 15 levels of the mine. Described mining experience relates to stopes above 9 level, where detailed structural information is not available. These areas are presently inaccessible.

Experience in cut-and-fill stopes between 11 and 15 levels suggests a mean stable span of about 3 - 5 metres. These estimates are tentative because of the difficulty associated with the extrapolation of experience gained in cut-and-fill stopes where hanging-walls are progressively disturbed during mining to sublevel open stoping situations.

THE MODIFIED-RMR RATING AND THE Q-INDEX ARE CONSIDERED TO BE POTENTIALLY USEFUL TOOLS IN PROVIDING FIRST-ORDER ESTIMATES FOR STABLE HANGING-WALL SPANS.

The two classification systems may be tailored to suit more closely local mining conditions.

8.5 Finite Element Predictions

Material properties used in the finite element analysis have been estimated on the basis of various rock mass classification systems. The results of the analysis may be anticipated to reflect the limitations inherent in these classifications.

However, there is a marked difference between the rock mass classification and finite element approaches. In the latter, the continuum around the opening is discretized into finite elements. This permits a definition of critical areas around the stope and enables
the assessment to be expressed in terms of the severity of ground collapses. Also, in contrast to the classification approach, the rock mass is modelled as a dynamic system. Excess loads in yielding elements are redistributed to surrounding areas.

THERE IS REASONABLE CORRELATION BETWEEN THE RESPONSE OF FINITE ELEMENT MODELS AND IN-SITU OBSERVATIONS.

Both sets of results are presented in Tables 10 - 13 and 15.

Considerable instability is suggested in hanging-walls and abutments of stopes within Orebody Nos. 5 and 7 at strike spans exceeding 40 metres. Stopes with spans of 25 - 30 metres are generally stable, apart from minor, local instabilities.

As noted earlier, a marked discrepancy is apparent between model and prototype for stopes in Orebody No. 11.

Monitored hanging-wall and footwall displacements do not provide sufficient data to validate rock mass modulus values assumed in the analysis. Evidence is contradictory.

However, it should be noted that the geology of monitored hanging-walls differs from that of the modelled stopes. The more massive silica-dolomite rock type is present within the walls. This imparts a greater stiffness to them and may account for the smaller deflections that have been measured in-situ. Displacements are also extremely sensitive to variations in the magnitude of the pre-mining stress field. Although inferences made without supporting field evidence can only be regarded as speculative, this variation alone can explain the observed differences in displacement.

It is interesting to note that Bridges (1979a) suggested a 50 - 60 mm displacement for the shale footwall of the stope located in Orebody No. 8. The pre-mining stress field of the area is complicated by the shadow or shielding effect of nearby mining. Estimates of the stress
acting normal to bedding range from 20 MPa to 35 MPa. The stress is also non-uniform across the stope. In view of the uncertainties, the predicted displacement of 75 - 80 mm (as extrapolated from Figure 51 for a hanging-wall span of 80 metres at a stress of 36 MPa acting normal to bedding) compares favourably with field observations. Full agreement is indicated for a normal stress of 26 MPa.

8.6 Probability of Failure Analysis Predictions

The analysis suggests that the probability of local failure and its extent within the hanging-wall and abutment areas of stopes in Orebody Nos. 5 and 7 are low for strike spans not exceeding 25 metres. Failure probabilities increase markedly as spans are extended to 40 and 55 metres.

These results are in good accord with field observations.

The significance of different pre-mining stress fields on the probability of failure around a stope are demonstrated in Figures 60 - 63. These plots clearly highlight the importance of the stress parameter in stope stability analyses and in the interpretation of past mining behaviour.
The thesis presents a systematic approach for the structural characterization of rock masses, estimation of material properties and the stability assessment of engineering structures within heterogeneous continua.

This chapter summarizes and discusses the conceptual approach used, basic principles, limitations and possible improvements to the model.

9.1 Background Considerations

Most rock masses are complex physical systems. The objective of any stability analysis is to achieve an understanding of their response under a set of specified external conditions.

A number of factors dictate the approach adopted to analyses. Two of the more important ones are:

(1) Physical dimensions of the rock mass block.

(2) Extent to which the rock mass block is transected by geological discontinuities.

Where block dimensions are relatively small and critically orientated weakness planes completely penetrate the mass, the response of the systems under load will be largely governed by the shear and stiffness characteristics of these discontinuities. Simple graphical techniques, similar to those applied by Miller and Barrett (1979) to
slender pillars, may be employed in the stability analysis. However, care should be exercised to ensure that the failure mode will be by movement along these planes and that it will not involve significant rotation and loss of interlock between the sub-blocks delineated by them. The latter mode of deformation may considerably reduce estimated shear strength of the mass (Ladanyi and Archambault, 1969).

Larger blocks with several sets of non-penetrating and non-uniformly distributed sets of weakness planes constitute more complex systems.

Adequate characterization of such systems is not an easy task. It demands the ability to quantify large volumes of structural data into a meaningful and useful form. Sufficient detail should be preserved in the model to reflect the inherent local variability in the system.

However, it must be borne in mind that the analysis of complex systems is difficult. Even where such analyses are numerically possible, the computational effort involved in reaching a solution may not be justified economically.

The tendency has been to reduce the number of components in systems. This simplifies the model. The approach has invariably resulted in the rock mass being characterized by a relatively homogeneous continuum or discontinuum. Directional properties common to all elements of such models are sometimes included.

Since the mid-1960's, large numbers of such parametric studies have been reported in technical literature. Modelling includes both physical and numerical techniques.
9.2 Basis of Adopted Model

In the present study, the rock mass is characterized as a heterogeneous continuum with statistically variable properties. The material model is a function of the inherent structural variability within the mass.

The basis of the model rests on the contention that any large block of rock mass may be represented by an equivalent system of smaller sub-blocks or modules.

9.3 Derivation of Model Parameters

A model for structural variability within the rock mass is derived on the basis of area-mapping of underground openings. Assessment of data involves an examination of the variability that exists between "unit areas" of the rock mass. Some assumptions are necessary to extend the model into the third dimension.

Several sets of weakness planes are defined and a statistically-based model is formulated to account for their spatial distribution in the mass.

The work undertaken at the Mount Isa Mine demonstrates that the spatial distribution of each set of fractures is not random. They tend to occur in zones or clusters. The extent of the zones in directions parallel and normal to the orientation of the average fracture plane of a set may be described statistically.
The use of standard semi-variogram techniques, such as presented in Clark (1979), to describe the extent of fracture zones was considered. However, the multi-component nature of the model and characteristics of the statistical distributions describing local variability in structure indicate that this approach would not offer any simpler solution than the one adopted.

The assumed structural model is validated by a computer program developed for the simulation of fracture distributions within rock mass blocks. Good correlation was observed between model and prototype. The model is extended to permit assessment of structural variability between rock mass blocks of any dimensions.

A number of published rock mass classifications are used to derive estimates of material properties. A computer program was developed to expedite the assessment. The analysis centres on "Monte Carlo" sampling of probability density functions for the intensity of each fracture set as well as other parameters and the derivation of appropriate classification ratings for several systems.

In theory, there is no limit to the complexity of the material model that may be assumed for individual modules of the rock mass. Direct field testing of large samples offers the most reliable solution. Alternatively, the results of physical modelling tests on blocky systems, such as described in Brown (1970a, 1970b), Einstein et al. (1969), Rosenblad (1970) and many others, may be employed to derive more realistic directional and stress-dependent continuum properties.

However, there are practical constraints. These largely arise from inherent limitations of the numerical techniques used to perform stability analyses. Although three-dimensional techniques have been available for several years, their widespread use is limited by cost considerations. Most analyses are performed on two-dimensional idealizations of problems. Currently available programs only permit relatively simple anisotropic materials to be assessed. It is anticipated that some of these restrictions may eventually disappear with the development of multi-laminate material models, such as
described by Zienkiewicz and Pande (1977) for a two-dimensional analysis. However, limitations are present at the moment and they dictate the complexity that may be assumed for material properties.

9.4 Stability Analysis

Three different approaches are used to assess the stability of stopes:

(1) Rock Mass Classifications

(2) Two-dimensional, Finite Elements, and

(3) Probability of Failure Analysis.

Results indicate a reasonable correlation with prototype response.

Certain difficulties are associated with each of the three analysis techniques employed.

None of the published rock mass classification systems is completely satisfactory. Estimates are conservative. However, in some instances, they are within 20 - 30 per cent of the values suggested by past mining experience in the silver-lead-zinc orebodies at Mount Isa Mine. Thus, a number of the classifications are considered to be potentially useful in providing first-order estimates. These systems may be tailored to suit more closely local mining conditions.

Limited evidence suggests a significant difference in the response between finite element analyses of models with homogeneous and statistically heterogeneous material properties. Differences are
apparent in the extent of plastic yielding and the distribution of stresses around the opening.

It is acknowledged that some of the observed differences may have arisen purely from the deficiencies that are inherent in numerical techniques, such as the "peculiarities of the discretization process" referred to in Baecher and Ingra (1979). These include the shape, size and orientation of elements in the mesh. However, because of the extent of discretization and the isoparametric nature of elements employed in the present analysis, it is unlikely that any significant portion of the observed difference can be attributed to this cause. The differences arise from the variation in local stiffness and strength of the continuum, and the redistribution of excess loads in yielding components of the mass.

Application of finite elements to analysis of statistically heterogeneous models is limited by cost considerations. These are prohibitive even when a relatively small number of models with elasto-plastic behaviour are involved. This problem is compounded in three-dimensional analyses.

The cost factor militates against the use of statistically-based finite element modelling approaches.

An alternative approach to assessing the potential extent of ground collapse around underground openings is offered by the "probability of failure analysis". This method has many advantages and it is a powerful tool for gaining a first-order appreciation of rock mass response. It can be readily employed in parametric evaluations of ground behaviour.
9.5 Conclusions

(1) All stability analyses are conceptual processes. They involve the idealization of real rock mass systems by means of simplified models.

(2) Most rock masses are complex and locally variable physical systems. They cannot be adequately represented by homogeneous models.

(3) The complexity of a rock mass system is reduced by the subdivision of the mass into an equivalent system of smaller modules or subsystems. This approach offers a sound framework for the statistical characterization of rock masses.

(4) The response of the whole system is equal to the total sum of the responses of individual subsystems that are located along "critical paths" within the system.

(5) Spatial distribution of fracture plane centres is not random in at least one rock mass. Members of each set of geological structures appear to occur in zones. However, the spatial distribution of "high intensity" zones is assumed to be random.

(6) "Area mapping" of underground openings offers a suitable method for the determination of statistically-based model for local variability in the intensity of geological structures.

(7) The commonly recommended method of single "line sampling" does not furnish sufficient data for the 2-D or 3-D characterization of rock masses.

(8) There are considerable difficulties associated in the estimation of rock mass properties such as unconfined compressive and tensile strengths, cohesion, friction angle,
modulus of deformation and Poisson's ratio.

(9) In-situ or laboratory testing of large rock mass samples offers the only reliable method for the determination of material properties.

(10) Reasonable, first-order estimates of some rock mass properties may be deduced for the dolomitic shales at the Mount Isa Mine on the basis of various published rock mass classification systems.

(11) Aspects of the following classifications appear to be potentially useful in the determination of rock mass properties:

**Strength:**
- (i) Hansagi's C-Factor in conjunction with Bieniawski's RMR-index.
- (ii) Deere's RQD-index, or
- (ii) Bieniawski's RMR-index.

(12) There are serious limitations inherent in the rock mass classification approach. Most of these centre on the simplicity of the ensuing material model and the validity of extrapolating relationships established at other sites to the mass under investigation.

(13) More complex material properties may be deduced for rock mass subsystems with specified structural configurations on the basis of numerous physical and numerical modelling studies of blocky systems already reported in technical literature.

(14) Practical constraints are commonly imposed on the complexity of the material model. These arise from limitations that are inherent in the numerical techniques used to perform stability analyses.

(15) The results of stability analyses undertaken in the present study suggest a reasonable degree of correlation between prototype and model response.
None of the published rock mass classification systems appears to be completely satisfactory with respect to the determination of stable hanging-wall spans. Estimates are generally conservative by at least 20-30 per cent.

Some classifications may be tailored to suit local mining requirements. The most promising classification systems are Barton's Q-index and Laubscher and Taylor's Modified RMR-index.

The limited evidence that is available suggests that there is a statistically significant difference between the response of finite element models with homogeneous and heterogeneous material properties.

Stability analyses are extremely sensitive to variations in the pre-mining stress field. Therefore, any correlation between model predictions and actual field observations will remain in the realm of speculation until field stresses are defined with some confidence.

The scope of any finite element analysis involving statistical assessment of heterogeneous models is markedly limited by the enormous computational effort associated with such studies. Even limited analyses involving 2-D models are outside the scope of routine assessments. 3-D analyses are prohibitive.

Assessment of past mining behaviour of stopes at the Mount Isa Mine is confounded by various practical difficulties. These are detailed in Paragraph 8.1 of the Ph.D thesis.

The "Probability of Failure" analysis offers a relatively simple, inexpensive and statistically-based method for first-order estimates of rock mass stability. Very few limitations are imposed on the complexity of the material model that may be assumed for rock mass properties.
(23) The "Probability of Failure" analysis approach may be extended to permit an approximate assessment of the extent of progressive collapse around underground openings.

(24) All phases of data assessment, statistical model formulation and verification, and stability analysis may be greatly expedited by the routine use of computer capabilities.

9.6 **Recommendations**

(1) The procedure of "Area Mapping" should be introduced as part of routine geotechnical investigations. This will permit the formulation of statistically-based models for the 2-D and 3-D variability in geological structure within rock masses. However, because of the associated work-load, this mapping should be restricted to selected areas of mine openings or surface excavations.

(2) Some form of the "Probability of Failure" analysis technique should be adopted for first-order evaluation of potential rock mass response. Such analyses will be markedly expedited by the development of the appropriate computer software. This will permit the procedure to be employed on a more routine basis as an aid to mine planning.

(3) Published technical literature on numerical and physical simulation of blocky systems should be reviewed critically and a model bank describing the response under load of subsystems with particular structural configurations should be established. This will furnish better material properties for statistically heterogeneous models of real rock masses.
Modelling predictions can only be validated by field observations. The art of predicting unsupported spans in rock masses will only be advanced by monitoring, documenting and interpreting the response of the mass during mining. This work should be undertaken on a more routine basis and the results should be made available in the form of a general data-bank.
REFERENCES


ADDITIONAL REFERENCES

This section includes additional references that have been cited in Baczynski (1977c, 1978b, 1979 and 1980b); technical reports submitted to the University of Melbourne as part of the Ph.D project.


Perkins, W.C. (1972). - Results of Confirmatory Drilling: 
Racecourse Orebodies, 13 Level to 15 Level. 


APPENDIX I - ROCK MASS CLASSIFICATION INPUT PARAMETERS

Al.1 Bieniawski's RMR Method

(i) Unconfined Compressive Strength of Intact Cores: 185 MPa

(ii) Rock Quality Designation (RQD): determined on basis of total fracture intensity in accordance with relationship presented in Figure 32 of the thesis.

(iii) Spacing of Joints or other Discontinuities: determined on basis of statistical model presented in Figure 27. A weighted rating was determined on basis of the relative frequency (i.e., intensity) of each fracture set.

(iv) Condition of Joints:
   (a) Bedding Plane Partings: relative frequency of graphite coated surfaces determined on basis of statistical model presented in Paragraph 3.3.4 of the thesis. These surfaces are classed as "slikensided surfaces" with a rating of 6. Non-graphitic surfaces were classed as "slightly rough surfaces, separation less than 1 mm, hard joint wall rock" with a rating of 20.
   (b) Other Fractures: same classification as adopted for non-graphitic bedding plane partings. A weighted average was determined on basis of the relative frequency of each discontinuity type.
(v) **Orientation of Joints:** each set of discontinuities as defined in Table 1 on page 33 of the thesis were classed individually as indicated below:

<table>
<thead>
<tr>
<th>Set No.</th>
<th>Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Very Favourable</td>
</tr>
<tr>
<td>2</td>
<td>Fair</td>
</tr>
<tr>
<td>3</td>
<td>Very Unfavourable</td>
</tr>
<tr>
<td>4</td>
<td>Very Unfavourable</td>
</tr>
<tr>
<td>5</td>
<td>Very Unfavourable</td>
</tr>
<tr>
<td>6</td>
<td>Unfavourable</td>
</tr>
<tr>
<td>7</td>
<td>Very Unfavourable</td>
</tr>
<tr>
<td>8</td>
<td>Unfavourable</td>
</tr>
<tr>
<td>9</td>
<td>Fair</td>
</tr>
<tr>
<td>10</td>
<td>Fair</td>
</tr>
<tr>
<td>11</td>
<td>Very Unfavourable</td>
</tr>
</tbody>
</table>

A weighted average rating was determined on the basis of relative frequency of each discontinuity set.

(vi) **Groundwater:** it was assumed that water was essentially absent, i.e., rock mass was "completely dry".

---

Laubscher and Taylor's Modified RMR Method

(i) **Unconfined Compressive Strength of Intact Cores:** as in Paragraph Al.1 above.

(ii) **Rock Quality Designation (RQD):** as in Paragraph Al.1 above.

(iii) **Spacing of Joints or Discontinuities:** as in Paragraph Al.1 above.

(iv) **Condition of Joints:**

(a) **Bedding Plane Partings:** frequency of graphite coated surfaces as in Paragraph Al.1 above. Graphitic surfaces
were classed as "(A) straight, (B) polished, (D) fine soft-sheared", whereas non-graphitic surfaces were assumed as "(A) straight, (B) smooth".

(b) All Other Fractures: same classification as adopted for non-graphitic bedding plane partings. A weighted average rating was determined on the basis of the relative frequency of each discontinuity type.

(v) Groundwater: as in Paragraph Al.l above.

(vi) Adjustments:

(a) Weathering: It was assumed that RQD and unconfined compressive strength were not affected (i.e., adjustment ratio of 1.0), whereas condition of joints was slightly affected (i.e., adjustment ratio of 0.95).

(b) Field and Induced Stresses: effect of stress is assumed to be dependent on the orientation of discontinuities within the rock mass. The following adjustment ratios were applied to the sets:

<table>
<thead>
<tr>
<th>Set No.</th>
<th>Adjustment Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,2</td>
<td>1.2</td>
</tr>
<tr>
<td>9,10,11</td>
<td>1.0</td>
</tr>
<tr>
<td>4,5,7</td>
<td>0.9</td>
</tr>
<tr>
<td>3</td>
<td>0.8</td>
</tr>
<tr>
<td>6,8</td>
<td>0.76</td>
</tr>
</tbody>
</table>

A weighted average was applied to the "condition of joints" parameter on the basis of the relative frequency of each set of discontinuities within the mass.

(c) Changes in Stress: It was not possible to evaluate the effect of this parameter on the basis of available information. A null adjustment factor of 1.0 was applied.

(d) Strike and Dip Orientation of Fractures: all discontinuities apart from set No.1 were assumed to be inclined away from vertical. The appropriate adjustment was determined with respect to the number of fracture sets that were assigned to the hanging-wall.

(e) Mining Technique: A classification just below "Good Conventional Blasting" was assumed for the mine, with
an adjustment ratio of 0.9. This ratio was applied to the RQD and Condition of Joints parameters in accordance with the maximum possible, i.e., each of the two parameters was reduced by 0.7 and 1.26 per cent, respectively.

Al.3 Barton's Q-Method

(i) Rock Quality Designation (RQD): as in Paragraph Al.1 above.

(ii) Joint Set Number: all discontinuities with an average intensity exceeding 1.0 m²/m³ were considered as sets, otherwise they were assumed to be random. No other adjustments (e.g., as for portals or tunnel intersections) were applied to the rating.

(iii) Joint Roughness Number: the following classifications and ratings were assumed for each fracture set:

<table>
<thead>
<tr>
<th>Set No.</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-10</td>
<td>(A) Discontinuous joints</td>
</tr>
<tr>
<td>11</td>
<td>(a) graphitic</td>
</tr>
<tr>
<td></td>
<td>(G) Slikensided, planar</td>
</tr>
<tr>
<td></td>
<td>(b) non-graphitic</td>
</tr>
<tr>
<td></td>
<td>(A) Discontinuous joints</td>
</tr>
</tbody>
</table>

A weighted rating was determined on the basis of the relative frequency of each set.

(iv) Joint Alteration Number: all fracture sets and non-graphitic bedding plane partings were classed as "(B) unaltered joint walls, surface staining only", whereas graphitic bedding plane partings were assumed as "(E) softening or low friction clay mineral coating, i.e., kaolinite, mica, or chlorite, talc, gypsum, graphite, etc. ....".

(v) Joint Water Reduction Factor: Class(A) was assumed, i.e., "dry excavations or minor inflow ....".
(vi) **Stress Reduction Factor:**

(a) **Orebody Nos. 5 and 7:** Items J and K of classification table were assumed to apply, i.e., the value of SRF is located in the range of about 0.75 - 2.0

(b) **Orebody No. 11:** Items F and G of classification table were assumed to apply, i.e., SRF is in the range of 2.0 - 5.0

(vii) **Excavation Support Ratio (ESR):** an ESR value of 5.0 is assumed to be representative of sub-level open stopes.
Minerva Access is the Institutional Repository of The University of Melbourne

**Author/s:**
Baczynski, Norbert Richard Przemyslaw

**Title:**
Rock mass characterization and its application to assessment of unsupported underground openings

**Date:**
1980

**Citation:**

**Publication Status:**
Unpublished

**Persistent Link:**
http://hdl.handle.net/11343/35714

**File Description:**
Rock mass characterisation and its application to assessment of unsupported underground openings

**Terms and Conditions:**
Terms and Conditions: Copyright in works deposited in Minerva Access is retained by the copyright owner. The work may not be altered without permission from the copyright owner. Readers may only download, print and save electronic copies of whole works for their own personal non-commercial use. Any use that exceeds these limits requires permission from the copyright owner. Attribution is essential when quoting or paraphrasing from these works.