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Underground metal mine crown pillar stability analysis

M. Tavakoli
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UNDERGROUND METAL MINE
CROWN PILLAR
STABILITY ANALYSIS

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

DOCTOR OF PHILOSOPHY

from

UNIVERSITY OF WOLLONGONG

by

M. Tavakoli, B.Sc., M.Sc.

Department of Civil and Mining Engineering
1994
AFFIRMATION

I hereby certify that the work presented in this thesis was carried out in the Department of Civil and Mining Engineering of the University of Wollongong and has not been submitted for any other degree.

M. Tavakoli
ABSTRACT

The open stope mining method is the most common underground extractive technique used in Australian metalliferous mines. The crown pillar as it stands in the vertical plane between two open stopes is an integral part of the global stability of an underground metal mine. The stability of crown pillars are significantly affected by the mechanical and physical properties of the rock mass, structural weaknesses, the initial state of the horizontal stress and the geometry of the crown pillar.

To date no integrated design methodology is available in the public domain and generally crown pillar design relies on past experience and rule of thumb. In this thesis the objective was to develop a method which will aid engineers in designing the optimum crown pillar at any mine no matter what the local conditions. To achieve this objective various methods, including empirical, numerical and theoretical methods of crown pillar design were investigated. This part of the study was used as a guide-line for modification of the available techniques and development of a complete design method for crown pillars.

In chapter 3 the voussoir beam and tributary area theories were modified. Using the modified versions a combined empirical and theoretical method of crown pillar design was developed. This method allows the engineer to determine an initial value for pillar span and thickness for the given conditions, and also to get a first estimate of the stress level in the pillar. Also in this chapter is a review of the work on stope design of Mathews et al (1981) and Potvin et al (1989). Finally a preliminary design methodology is presented.

Chapter 4 has two basic sections dealing with the techniques used for gathering and reducing field and laboratory data essential for design. The first section deals with the techniques, such as scanline surveying, used in the field to determine the structure of a rock mass. The second describes the various tests which were conducted to determine the mechanical properties of the rock mass.

Crown pillar stability assessment based on data collected from case studies of a copper mine (CSA Mine, Cobar) and a lead-zinc mine (NBHC Mine), was carried out in chapters five and six. CSA Mine, NSW, Australia was chosen as the first site for evaluation of the stability of crown pillars. Cobar is a copper mine where open stoping operations are carried out in several parallel orebodies which dip between 75° to 85° and have an average thickness of 12 m. The NBHC Mine, Broken Hill, NSW, Australia was the second mine.
chosen for a crown pillar case study. In this mine the orebody has been formed from several massive and thick discrete lodes or lenses. The geometries of the stopes and crown pillars are much more complicated than at CSA Mine and the stress distribution due to mining activities is also complex. The crown pillar span varies from one area to another; this variation being influenced by the grade of ore and the geometry of the orebody.

The results from the joint surveys and rock tests were used as data in order to determine the applicability of the various design methods and back analyse stable and failed pillars in these mines. Part of the back analysis included the use of UDEC (a distinct element program) to simulate the action of the crown pillars under various stress regimes and various mining sequences. To gain a better understanding of the failure mechanism and modelling capabilities of UDEC, a series of parametric studies were also carried out. Finally, after comparing the different methods, conclusions are drawn and a methodology for crown pillar design is suggested and recommendations for future work are given.
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This thesis is dedicated to my wife who helped me spiritually and mentally during the four years of hard work which it took to complete this thesis. Without her assistance I would not have been able to finish this work on time.
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CHAPTER 7

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<thead>
<tr>
<th>Abbreviation</th>
<th>Full Form</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABET</td>
<td>Accreditation Board for Engineering and Technology</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
</tr>
<tr>
<td>Brg.</td>
<td>Bearing</td>
</tr>
<tr>
<td>C</td>
<td>Cohesion</td>
</tr>
<tr>
<td>CAF</td>
<td>Cut And Fill</td>
</tr>
<tr>
<td>Con.</td>
<td>Constitutive</td>
</tr>
<tr>
<td>CSIRO</td>
<td>Commonwealth Scientific and Industrial Research Organisation</td>
</tr>
<tr>
<td>De</td>
<td>Equivalent Dimension</td>
</tr>
<tr>
<td>DRMS</td>
<td>Design Rock Mass Strength</td>
</tr>
<tr>
<td>E</td>
<td>Elastic modulus</td>
</tr>
<tr>
<td>ESR</td>
<td>Excavation Support Ratio</td>
</tr>
<tr>
<td>FLAC</td>
<td>Fast Lagrangian Analysis of Continua</td>
</tr>
<tr>
<td>FW</td>
<td>Footwall</td>
</tr>
<tr>
<td>HW</td>
<td>Hangingwall</td>
</tr>
<tr>
<td>IRS</td>
<td>Intact Roc Strength</td>
</tr>
<tr>
<td>ISRM</td>
<td>International Society of Rock Mechanics</td>
</tr>
<tr>
<td>L/D</td>
<td>Length to diameter ratio</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Differential Transducer</td>
</tr>
<tr>
<td>K</td>
<td>Horizontal to vertical stress ratio</td>
</tr>
<tr>
<td>Kn</td>
<td>Joint normal stiffness</td>
</tr>
<tr>
<td>Ks</td>
<td>Joint shear stiffness</td>
</tr>
<tr>
<td>Max</td>
<td>Maximum</td>
</tr>
<tr>
<td>Min</td>
<td>Minimum</td>
</tr>
<tr>
<td>MRMR</td>
<td>Modified Rock Mass Rating</td>
</tr>
<tr>
<td>NBH</td>
<td>New Broken Hill</td>
</tr>
<tr>
<td>NGI</td>
<td>Norwegian Geotechnical Institute</td>
</tr>
<tr>
<td>No.</td>
<td>Number</td>
</tr>
<tr>
<td>NA</td>
<td>Not available</td>
</tr>
<tr>
<td>NR</td>
<td>Not required</td>
</tr>
<tr>
<td>RQD</td>
<td>Rock Quality Designation</td>
</tr>
<tr>
<td>RMR</td>
<td>Rock Mass Rating</td>
</tr>
<tr>
<td>RSR</td>
<td>Rock Structure Rating</td>
</tr>
<tr>
<td>RMS</td>
<td>Rock Mass Strength</td>
</tr>
<tr>
<td>SMEC</td>
<td>Snowy Mountain Engineering Corporation</td>
</tr>
<tr>
<td>UCS</td>
<td>unconfined Compressive Strength</td>
</tr>
<tr>
<td>UDEC</td>
<td>Universal Distinct Element Code</td>
</tr>
</tbody>
</table>
## List of Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_v$</td>
<td>Vertical stress, Mpa</td>
</tr>
<tr>
<td>$\sigma_h$</td>
<td>Horizontal stress, Mpa</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson's ratio</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Density, Kg/m$^3$</td>
</tr>
<tr>
<td>$g$</td>
<td>Gravity, m/s$^2$</td>
</tr>
<tr>
<td>$K$</td>
<td>Bulk modulus, GPa</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus, GPa</td>
</tr>
<tr>
<td>$E$</td>
<td>Elastic modulus, GPa</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson's ratio</td>
</tr>
<tr>
<td>$P$</td>
<td>Rock load, KN per unit length of tunnel</td>
</tr>
<tr>
<td>$B$</td>
<td>Tunnel width, m</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Density of rock, kg/m$^3$</td>
</tr>
<tr>
<td>$h$</td>
<td>Rock load height, m</td>
</tr>
<tr>
<td>$J_n$</td>
<td>Joint set number</td>
</tr>
<tr>
<td>$J_r$</td>
<td>Joint roughness number</td>
</tr>
<tr>
<td>$J_a$</td>
<td>Joint alteration number</td>
</tr>
<tr>
<td>$J_w$</td>
<td>Joint water reduction number</td>
</tr>
<tr>
<td>$L$</td>
<td>Roof span, m</td>
</tr>
<tr>
<td>$t$</td>
<td>Roof thickness, m</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Weight density of the rock, MN/m$^3$</td>
</tr>
<tr>
<td>$\delta_{\text{max}}$</td>
<td>Maximum deflection, mm</td>
</tr>
<tr>
<td>$\tau_{\text{max}}$</td>
<td>Maximum shear stress, Mpa</td>
</tr>
<tr>
<td>$\sigma_{\text{max}}$</td>
<td>Maximum tensile stress, MPa</td>
</tr>
<tr>
<td>$M$</td>
<td>Maximum moment in the beam, N m</td>
</tr>
<tr>
<td>$W$</td>
<td>Load on each unit length of beam, N/m</td>
</tr>
<tr>
<td>$I$</td>
<td>Second moment of roof beam section (moment of inertia), m$^4$</td>
</tr>
<tr>
<td>$C$</td>
<td>Distance from the centre of gravity to the extreme fibre, m</td>
</tr>
<tr>
<td>$b$</td>
<td>Width of the roof beam, m</td>
</tr>
<tr>
<td>$d$</td>
<td>Roof beam thickness, m (page 42)</td>
</tr>
<tr>
<td>$t$</td>
<td>Thickness of the beam, m (page 42)</td>
</tr>
<tr>
<td>$\sigma_x$</td>
<td>Resultant stress due to prestress, (MPa)</td>
</tr>
<tr>
<td>$F$</td>
<td>Factor of safety (page 42)</td>
</tr>
<tr>
<td>$S$</td>
<td>Span of beam, m (page 44)</td>
</tr>
<tr>
<td>$W$</td>
<td>Roof beam total load, MN/m$^3$</td>
</tr>
<tr>
<td>$R$</td>
<td>Modulus of rupture of the roof rock (tensile strength in flexure), MPa</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
</tr>
<tr>
<td>$\sigma_p$</td>
<td>Pillar load or the average pillar stress, MPa</td>
</tr>
<tr>
<td>$\sigma_v$</td>
<td>Virgin vertical stress, MPa</td>
</tr>
<tr>
<td>$H$</td>
<td>Depth below surface, m</td>
</tr>
<tr>
<td>$W_p$</td>
<td>Width of Pillar, m</td>
</tr>
<tr>
<td>$W_0$</td>
<td>Width of opening, m</td>
</tr>
<tr>
<td>$e$</td>
<td>Extraction ratio</td>
</tr>
<tr>
<td>$\sigma_0$</td>
<td>Pre-mining stress normal to orebody, MPa</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Angle of orebody inclination, degrees</td>
</tr>
<tr>
<td>$\sigma_h$</td>
<td>Horizontal stress, MPa</td>
</tr>
<tr>
<td>$W$</td>
<td>Pillar width, m</td>
</tr>
<tr>
<td>$h$</td>
<td>Pillar height, m</td>
</tr>
<tr>
<td>$\sigma_l$</td>
<td>Strength of a cubical specimen of critical size or greater</td>
</tr>
<tr>
<td>$\sigma_c$</td>
<td>Uniaxial compressive strength</td>
</tr>
<tr>
<td>$b$</td>
<td>Triaxial stress factor (4 for coal)</td>
</tr>
<tr>
<td>$k$</td>
<td>Strength of a foot cube (page 70)</td>
</tr>
<tr>
<td>'a' and 'b'</td>
<td>Constant (page 70)</td>
</tr>
<tr>
<td>$n$</td>
<td>Ratio of roof beam section under horizontal compression</td>
</tr>
<tr>
<td>$f_c$</td>
<td>Maximum lateral compressive stress, MPa</td>
</tr>
<tr>
<td>$W$</td>
<td>Roof beam weight, MN</td>
</tr>
<tr>
<td>$z_0$</td>
<td>Internal moment of arm of the couple of forces, m</td>
</tr>
<tr>
<td>$W$</td>
<td>Roof beam weight with unit length, MN (page 74)</td>
</tr>
<tr>
<td>$nt$</td>
<td>Load depth</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Friction angle, degrees</td>
</tr>
<tr>
<td>$A$</td>
<td>Cross sectional area, m$^2$</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson’s Ratio</td>
</tr>
<tr>
<td>$W$</td>
<td>Weight of sliding wedge (page 112)</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Angle of inclination of discontinuity to horizontal, degrees</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Angle between rockbolt and the plane of discontinuity, degrees</td>
</tr>
<tr>
<td>$K$</td>
<td>Stiffness of the material, N/mm</td>
</tr>
<tr>
<td>$\varepsilon_a$</td>
<td>Axial strain</td>
</tr>
<tr>
<td>$\varepsilon_d$</td>
<td>Diametric strain</td>
</tr>
<tr>
<td>$P$</td>
<td>The load required to break the specimen, KN (page 121)</td>
</tr>
<tr>
<td>$D$</td>
<td>The diameter of the rock specimen, mm (page 121)</td>
</tr>
<tr>
<td>$I_s$</td>
<td>The point load strength index, MPa</td>
</tr>
<tr>
<td>$m$</td>
<td>The slope of the straight line portion of the $\sigma_1$ versus $\sigma_3$ curve.</td>
</tr>
<tr>
<td>$b$</td>
<td>The intercept of the straight line portion with the $\sigma_1$ axis.</td>
</tr>
<tr>
<td>$\tan \phi$</td>
<td>Coefficient of friction of the shear surface</td>
</tr>
</tbody>
</table>
CHAPTER 1

INTRODUCTION, AIMS & SCOPE

1.1 Crown pillar definition and statement of the problem

A crown pillar generally is a horizontal part of an ore body between two stopes in a metalliferous mine. Crown and rib pillars are the main support structures for stopes during excavation. Although a "thick" crown pillar will provide good support for the hangingwall and aid with global stability, it may be inefficient from an economic viewpoint. As such the maximum amount of ore should be extracted from the primary stopes, to reduce the risk of sterilisation of reserves, thus reducing the thickness of the crown pillar. The optimisation of crown pillar dimension is very important for the metalliferous mining industry. Very large and thick crown pillars cause the loss of reserves whilst undersized pillars may cause failure and instability in the mine. The use of crown pillars to limit stope wall movement and reduce the possibility of large scale failure is common practice in open stope mining. Prediction of the optimum thickness of crown pillar is complex, generally based on practical experience with input from numerical analysis and various empirical techniques. The use of cable bolting to aid stability of the crown pillar can be useful by increasing the quantity of ore which could be extracted from the crown, thus increasing primary extraction.

The limit to the minimum thickness of pillar which will remain stable is a function of many parameters including; the insitu mechanical properties of the rock mass and the
bilateral forces acting on the pillar (due to the pillar self-weight and the re-distributed stressfield). Other factors such as mechanical support (cable or rock bolts) and backfilling may also be applicable to any analysis. Crown pillars should be designed so that they remain stable for a predetermined period of time. The time that crown pillars have to be stable may be the life time of the mine or, when backfilling, the duration of extraction of all the ore from the stope plus the time it takes the open stope to become stable with fill. However, increases in the depth of mining and new mining practices coupled with the need for extraction of valuable reserves highlights the need for a more quantitative approach to crown pillar design. In open-stope mining methods the cost of pillar recovery per ton of ore is usually much higher than that of mining primary stopes, therefore, maximisation of the size of primary stopes and minimisation of the size of rib and crown pillars is very important in planning the mining operation.

An acceptable theoretical method for the design of crown pillars in metalliferous mines has not yet been developed and because mining conditions are relatively complex, the application of empirical design rules and criteria is not always possible. To develop a particular method, the behaviour of stopes and pillars, and particularly failure behaviour together with information about the rock properties, structure and in situ stress conditions should be collected. To gather this information use of improved methods of measurement and monitoring of rock behaviour, and rock failure prediction is needed. Theoretical solutions for determination of the effects of the support of crown pillars by cable bolting is also not adequate, because most methods use some assumptions and simplifications which may be appropriate only in some conditions (Fuller, 1983). Consideration of these facts and addressing the problem of optimisation of the crown pillar thickness based on empirical, analytical and numerical methods is the subject of the research undertaken in this thesis. For this purpose the currently available methods of crown pillar design will be examined and a design method will be developed with consideration of all parameters affecting crown pillar stability. The developed method should be capable of designing a safe and economic crown pillar.

### 1.2 Research Scope

Although there has been some effort in recent years to document the procedures which have been used by different mines for designing crown pillars, existing design methods are generally limited in scope (Betournay, 1989), and as such there is still a need to create a general design method which will take account of the majority of mining situations. Furthermore, it is also recognised that existing analytical methods should be verified and extended to consider actual failure mechanisms of crown pillars. Unfortunately data about
the geometry of failed crown pillars is very limited. Some work has been done to create a
design methodology and guide-line for the design of crown pillars (Betournay, 1986,
Betournay, 1987, Betournay, 1989, Hoek, 1989) but still there is a significant lack of
information on this subject.

This thesis describes work which has been done to help with the design of stable crown
pillars in metalliferous mines. The preliminary stage of this thesis was a literature review
of available crown pillar design methods, the purpose of which was to determine if any of
the available design methods were suitable as the basis for further development. In
addition, two mines CSA Mine, Cobar and New Broken Hill Mine, Broken Hill were
visited to obtain data on rock mass properties and crown pillar geometry.

During data collection from the sites visited a joint survey was completed and rock
samples were collected for laboratory testing. The results of the joint surveys and rock
tests were used as data in order to determine the applicability of the various methods and
back analyse stable and failed pillars in these mines. For the back analysis empirical
methods were used for the evaluation of the maximum unsupported span and support
requirements for a stable span. Voussoir arch theory was applied for assessing the critical
span of crown pillars with different thicknesses, and UDEC (a distinct element program)
was used to simulate the action of crown pillars under various stress regimes and various
mining sequences.

The thesis concludes with a comparison of the results obtained from the various design
methods, and a recommended general method for all situations.

1.3 Design in engineering

In recent years it has been realised that a good designer not only should have technical
knowledge but must also know the principles of design. Technical knowledge of design
is used to develop various design solutions and for selecting the best among them.
Design principles are a systematic methodology that should be followed during technical
design. In the field of rock mechanics only a limited attempt has been made to describe
the importance of the principles of engineering design for rock mechanics (Bieniawski
1990, 1992). Engineering design has been described by the Accreditation Board for
Engineering and Technology (ABET 1987) as follows:
"Engineering design is a process of devising a system, component, or process to meet desired needs. It is a decision making process (often iterative), in which the basic sciences, mathematics, and engineering sciences are applied to convert resources optimally to meet a stated objective. Among the fundamental elements of the design process are the establishment of objectives and criteria, synthesis, construction, testing and evaluation. In addition, sociological, economic, aesthetic, legal and ethical considerations need to be included in the design process"

1.3.1 Design theory

For engineering design it is necessary to know design theory and methodology. "Design theory is a systematic statement of principles and experimentally verified relationships that explain the design process and provide the fundamental understanding necessary to create a useful methodology for design" (Bieniawski, 1992). Six design principal were proposed by Bieniawski (1992). The basis of these design principles for evaluation and optimisation of alternative designs are as follows:

(1) Independence Principle: There are a minimum set of independent functional requirements that completely characterise the design objectives for a specific need.

(2) Minimum Uncertainty: The best design is the one which has minimum uncertainty about geological conditions.

(3) Simplicity Principle: For minimising the complexity of the design solution a minimum number of design components in relation to each functional requirement should be created. In brief this principle says "the simpler, the better".

(4) State-of-The-Art Principle: The best design maximises the technology transfer of the state-of-the-art research findings.

(5) Optimisation principle: The best design is the optimum which is a result of applying different designs based on optimisation theory and choosing the best of them based on quantitative evaluation.

(6) Constructibility Principle: The best design is the one that creates the most efficient construction in the rock by using the most appropriate construction methods and excavation sequences.
1.3.2 Design Methodology

The design methodology should be a guide-line or a check-list for the designer to reach the problem objective by applying the best design methods. It should be a sequence of steps which prevents confusion in the design process and also be a useful reference of where the project is at, where it should go and what is the next step in the design procedure. The design methodology for rock mechanics as proposed by Bieniawski (1992) is shown in Figure 1.1.

1.4 Open stope mining method

About two-thirds of production from underground mines in Australia is extracted by open stoping methods. In future this proportion will progressively increase and the conditions under which the method is applied will become more severe. Generally this method has been chosen because there are a range of problems and limitations in other mining methods. However, there is also evidence that the full potential of the method because of the followings limitation is not being achieved (Malcolm, 1982):

- Zones of poor ground conditions that delay access and development;
- Crown pillars that are irrecoverable;
- Substantial overbreak from the walls and backs of stopes;
- The lack of a widely accepted method for stope design.

Common terms used for open stoping are; open, sublevel, longhole and blasthole. The most common elements of open stoping can be summarised as follows (Malcolm, 1982):

- Fully open at some stage, without substantial collapse or caving;
- Extending from sublevel to sublevel, with operation only from these sublevels;
- Broken ore moves by gravity alone to drawpoints, which are fixed;
- Use of long blastholes for the blasting operation;
- Most blastholes are in sub-vertical planes, and most can be, but are not necessarily, drilled downwards;
- Open spans or strike is limited usually to tens of metres;
- The dip of the stope is usually more than 50°;
- An initial expansion room is created at the bottom or side of each stope;
- Miners do not work within the stope.
Chapter 1, Introduction, Aims and Scope

Figure 1.1 Engineering design principles for rock mechanics (after Bieniawski, 1992).
1.4.1 Stability of underground open stopes

Stability in mining can be divided into three different levels or categories; global, regional and local depending on the volume of rock involved. The three different levels of stability can be defined as follows (Stillberg, 1984):

(1) Global instability will occur if a major structure or part of it, for example, the hangingwall or crown pillar collapses and makes difficult the controlled mining process. This collapse must be prevented by reinforcing or designing crown and rib pillars so as to give the hangingwall sufficient support during extraction of the stopes.

(2) Regional stability must be considered when designing the stope height and pillar dimensions. Stability of the hangingwall is essential to prevent extensive failure which may result in serious dilution and possibly major global instability.

(3) Local stability is related to drill and loading levels, blast damage and unfavourable orientation of discontinuities which cause wedges or blocks to fall into the drifts and ore passes. The economic effect of this type of instability problem may not be dangerous to the mine structure, however, safety considerations should not be ignored.

1.4.2 Stress around underground openings

The design of the longhole sublevel mining method was originally based on the traditional assumption that the principal load is caused by the weight of the overlying rock, and that horizontal stress is equal to one third the vertical stress (Borg & Leijon, 1984). However, as mining progressed, the virgin lateral stress was found to be equal to or greater than the vertical stress.

The determination of the magnitude and direction of the insitu stress field around an underground excavation is an important step in the design of underground openings. There are four methods available for obtaining information on the insitu stress field (Duvall, 1976).

(1) The stress field can be estimated from gravity loading. The vertical and horizontal stresses in this case are estimated by the following Equation:

\[ \sigma_v = \rho \, gh \]  
\[ \sigma_h = \left( \frac{v \cdot \sigma_v}{1-v} \right) \]  

Eq. (1.1)  
Eq. (1.2)
Experience has shown that this technique gives a fairly accurate estimate of the vertical stress but usually underestimates the horizontal stress and it cannot predict the variation of horizontal stress with direction, which is the usual case. In Australia the maximum horizontal stress is greater than the vertical stress.

(2) The magnitude and direction of the horizontal stresses can be determined in a deep vertical drill hole by the hydro-fracturing technique (Obert and Duvall, 1976). This technique assumes that one of the principal stresses is known and it is towards the hole axis. This method gives fairly reliable results in rock masses in which the joints and fractures are not open and free. However, it is expensive and is not recommended for the preliminary steps in the design of underground openings.

(3) The horizontal stress field can be estimated from nearby sites and the vertical stress estimated by Equation 1.1.

(4) The horizontal stress field can be estimated in near-surface rocks to depths of 15 m to 25 m by the overcoring stress relief technique. Using this method the presence of horizontal tectonic stresses in a good sound rock near the surface can be estimated. If horizontal tectonic stresses are found in near-surface rocks, they most likely can be found in deeper rocks. Therefore, this information can be used to predict the magnitude and direction of the insitu stress field in deeper areas.

1.4.3 Effect of horizontal stress

Determination of horizontal stress is very important step in the stability assessment of underground openings. The thickness of the arch formed in the roof of underground openings depends mainly on the horizontal stress, roof span and the rock mass quality. The mode of failure also changes from shear to compressive failure or buckling with increasing horizontal stress. In the case of low horizontal stress, the height of the unstable
1.4.4 Concept of potential failure zones

Hoek (1976) described a failure concept which assumes brittle fracture behaviour of the rock. The following detail procedure must be followed when applying this concept:

(a) Calculate the elastic response of the rock mass to mining, in terms of stresses and/or strains.

(b) Determine a failure criterion for the rock mass considering the following stages:

(1) From the onset of rock fall problems before any roof support is supplied; mining can still be continued.
(2) After critical roof failure; mining must be stopped, or the mining procedure must be modified.

(c) Compare the elastic stresses/strains induced in the rock with the rock strength; expressed in terms of the failure criterion. The points at which the stress/strain values are equal to the values given by the failure criterion can be considered as the boundary of a potential failure zone.

(d) By comparison of the insitu observations obtained from the mine with predicted critical levels for the stopes the validity of the prediction can be checked.

1.4.5 Typical modes of failure in open stopes

The following factors may contribute to failure in open stopes:

- Size and geometry of openings.
- Geological features, such as folds, joints, faults and shears.
- Stress redistribution due to excavations and mining activities.

Failure will commonly occur as a result of a combination of these factors. For example when the size of an opening increases the possibility of failure for geological or stress reasons also increases. Most stability problems in stopes are related to discontinuities in the rock, and not to the rock itself. Poole and Mutton (1977) summarised the typical modes of failures in Cut And Fill (CAF) stopping as:
(1) Longitudinal wedge failure in the back.
(2) Small wedge failure in the back.
(3) Hangingwall failure; large vertical exposures of the stope hangingwall can fail, especially when it is composed of incompetent rock.
(4) Transverse wedge failure in the back.
(5) High stress induced failures; ground failure occurs due to high stress concentrations as a result of mining a number of orebodies simultaneously.

1.4.6 Support of underground openings

Adequate and efficient support of openings is essential for successful mining. Even when using modern technology, most underground mining accidents can be related to ground control and in such cases roof support is the most important consideration. Support is required to maintain the integrity of a rock mass so that the rock mass can support itself. Ideally an adequate support system should be designed and agreed upon before the development of an underground opening. The support system merely helps the rock to support itself and the surrounding rock takes the majority of the induced-mining stresses. The greatest stress concentration may be at the surface of the opening and this together with the effect of blasting may cause the surface of the opening to fail. Therefore, some kind of support may be necessary for the protection of miners from possible breakage of the roof and sidewalls.

Generally there are two methods of support for an underground opening namely active and passive support. Active support or rock reinforcement is a method in which supporting elements become an integrated part of the rock mass around the opening. Rock bolting is a good example of active support and it is the latest technological revolution in ground control. In passive support, supporting elements are external to the rock and respond to inward movement of rock surrounding the excavation. Steel sets are an example of this type of support (Hoek and Brown, 1980). A combination of these two methods may be recommended for particular rock mass classifications.

The initial support must be installed as soon as possible after excavation so that it can help the rock mass to remain intact (Hoek and Wood, 1988). The final support must be installed to bear the induced stress and stress changes during the life of excavation. The success of a support system depends on correct installation and the use of materials of the right quality. Experience has shown that simple systems which have been correctly installed are more helpful than complicated techniques where the possibility of error during installation is high.
In metalliferous mines the major function of ground support is to maintain the opening shape and size for further mining operations and the safety of workers. The continual evaluation of ground control is necessary, as in situ stresses and ground conditions change. Rock mass classification systems can be used for estimation of rock support requirements, however, they cannot predict the support of unstable isolated structural features such as blocks or wedges. When using these systems, if no systematic support is considered necessary, the possible need for support of isolated blocks or wedges should be considered.

1.5 Crown Pillar

A crown pillar is a horizontal part of an orebody which stands between two stopes to help maintain local and global stability of the rock mass. These mining structures when situated near the surface are called surface crown pillars and when situated at depth simply crown pillars. Figure 1.2 shows the location of surface crown pillars and crown pillars in an underground metal mine.

Numerous parameters affect the stability of a crown pillar. In general these parameters can be grouped in two sections; geological and mining.

Geological parameters:

- Dip of orebody.
- Rock types; hangingwall, footwall and orebody.
- Strength and deformation characteristics of hangingwall, footwall and orebody, as defined by rock mass classification.
- Geometry of multiple ore zones (if applicable).
- Virgin stress conditions.
- Properties of contact zones between ore and country rock.

Mining parameters:

- Geometry of crown pillar and surrounding stopes.
- Support methods (including backfilling).
- Mining sequence.
- Stress redistribution caused by mining.
Recently some efforts have been made to collect information about case studies and different design methods for these complex structures in hard rock. Betournay (1989) listed a number of publications related to crown pillars design since 1984. Furthermore a comprehensive investigation of surface crown pillar design methods was carried out by him. Failure mechanisms, stress conditions, in situ surveys, consideration of the three-dimensional volume of the crown pillar and numerical modelling are recognised as major components of pillar design (Betournay, 1989).

1.5.1 Method of evaluation of crown pillar stability

Structural geology as well as geometry of the excavation play a significant role in crown pillar stability, as the pillar failure mechanism can be affected by both. Due to the lack of precise geotechnical data for many cases in which failure has occurred, back analysis using various theoretical and empirical methods must be examined to obtain more information about the mechanism of failure. It is important to validate the results of theoretical or empirical solutions by comparing them with observations or records of actual failures.
1.5.2 Crown pillar failure mechanism

In past several different failure mechanisms have been used to assess the stability of crown pillars. Sarkka & Halonen (1984) used a modified Mohr-Coulomb failure criterion to identify unstable zones in the crown pillar. Von Kimmelmann and Hyde (1984) compared the strength and stress in the crown pillar to obtain a safety factor. For safety factors less than one the crown was assumed to be failed. For a given crown pillar geometry the geological structure may be the main cause of failure and sometimes it is important to identify particular geological weaknesses that may lead to instability. In other cases high horizontal stresses in the crown pillar control failure development. Individual major weakness planes such as faults may not be the main cause of failure in this situation as they have usually been recognised by the mining engineer and taken into consideration at the initial design stage. In blocky rock masses the intersection of several discontinuity sets may be the cause of failure.

For understanding the structural mode of failure stereonet diagrams which help to identify major discontinuity sets and possible failure geometries are useful. For this purpose data must be collected on spacing, continuity and orientation of major discontinuities and other properties of the rock mass surrounding the excavation. It is clear that in situ stresses, structural geology and the geometry of the crown pillar all play a significant role in the stability condition, and all can be used to help determine the possible mode of failure. Figure 1.3 shows examples of surface crown pillar failures which have been derived from some case studies.

As shown in Figure 1.3 in a potentially low stress environment such as surface crown pillars failure is expected to be controlled by structure rather than stress. Faults, shear zone and schistosity may occur to affect the stability. In massive rock localised degradation and readjustment of tensile stresses is expected (Figure 1.3, a). In a sound rock environment discontinuities are the most critical parameter (Figure 1.3, b and c). In altered rock, localised shear failure such as chimneying, crown degradation or large scale movement is expected (Figure 1.3 d, e and f). In all these cases it is critical to know the stress redistribution around the excavation and to know if this stress is enough to prevent direct gravity failure or sliding block failure (Betournay, 1989).

The result of investigations by Carter (1989) shows that most crown pillar failures have occurred as a result of sliding of adversely oriented joints in the rock mass at the hangingwall or footwall contact. Very few failures have been dominated by a major structural weakness such as a fault or cross-cutting shear zone. An important point is that
Figure 1.3 Examples of surface crown pillar failure (after Betournay, 1989)
assessment of stability using several design methods increases the reliability of the predicted results.

1.6 Research Objectives

The objective of the research was to develop a method which can be used in the design of crown pillars in underground metal mines. To achieve this objective the first stage was a literature review of rock mass classification systems, followed by a study of published work on underground metal mine support and crown pillar design methods. The second stage was field work which involved the analysis of case studies and data collection at particular mines; including methods of support and occurrence of failure. The next stage was a complete analysis of the case studies reviewed and the development of a combined analytical and empirical method for crown pillar design. Finally some suggestions and recommendations for further work are given. The objectives can be summarised as follows:

- To study the available design methods and determine whether they can be improved by considering past failure mechanisms.

- Identify the best design method by testing by back analysis whether or not this method can predict the failure mechanism of particular crown pillars.

- The final objective of the research was to produce the basis of a design methodology for the design of crown pillars in metal mines.

The procedure for reaching the objectives undertaken in this research is shown in Figure 1.4.
Figure 1.4 Procedure for reaching the objectives of the research.
CHAPTER 2

CROWN PILLAR DESIGN METHODS : A REVIEW

2.1 Introduction

Several analytical, empirical and numerical methods are available for assessing the stability of crown pillars. Each of these methods have some advantages and disadvantages. In this chapter the most common and efficient methods which can be used for the design of crown pillars will be discussed. Empirical methods such as rock mass classification can only provide general guide-lines for design and should be used with other methods for greater reliability. Among the available methods are analytical formulae such as those based on elastic beam theory, voussoir solutions and arch theory. Conventional numerical methods for rock mechanics applications include finite element, boundary element and distinct element. These different methods have been widely used for the design of underground excavations. Numerical modelling has been recognised as a powerful tool for solving the complex stability problems of crown pillars. So far most of the available information about crown pillar design has been based on single case studies.

At present crown pillar design in most mines has been based on past experience and 'rules of thumb'. In this study the factors which affect the stability of crown pillars will be investigated. Empirical, analytical and numerical methods are the most common tools which have been used for this purpose. Back analysis can be used to obtain a better understanding of the failure mechanisms and the level of stability or instability of past crown pillars.
2.2 **Empirical methods**

In most mines crown pillar design is only considered as an aid to choosing an appropriate pillar thickness to span ratio. Other dimensions of stopes such as the length or height are governed by other mining considerations. Past experience in the mine is the most important factor in this evaluation and this method of design is still used in many mine sites as a guideline. One reason for using a ‘rule of thumb’ for design purposes is the lack of widely acceptable methods for crown pillar design.

Most empirical methods give some guidelines to safe spans or thickness to span ratio, and even though theoretical justification of these methods is very difficult, they are still commonly used in mine design.

2.2.1 **Rock mass classification**

Rock mass classification provides a practical way for communication between geologists and engineers. An engineering classification is an attempt to assess the stability of a rock mass for a given project and consequently the selection of the parameters for such a classification is of special importance. In the case of a jointed rock mass a single parameter or index cannot completely describe the rock mass for all engineering purposes, therefore, a number of classification systems have been developed, each of which emphasises a particular property of the rock mass. Some systems assign numerical values for those properties which are considered to be effective in influencing the behaviour of the rock mass. The results from these systems may give an indication of the cavability, stand up time of unsupported spans, the support required for various spans and the stability of rock walls of dams or pits in relation to a particular rock mass quality. However, these classification systems have some deficiencies and must be used with extreme care. It is particularly important to know that a particular classification scheme may only give reliable results in circumstances similar to those for which it was originally developed. In brief the aims of the engineering classification of rock masses can be described as follows (Bieniawski, 1984):

a) to divide a particular rock mass into groups of similar behaviour;
b) to provide a basis for understanding the characteristics of each group;
c) to yield quantitative data for engineering design;
d) to provide a common basis for communication.
A classification system should also have the following characteristics (Bieniawski, 1984):

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

2.2.2 Rock mass classification review

There are many different classification systems available, the more widely recognised systems are highlighted below (Hoek and Brown, 1980) and listed in chronological order in Table 2.1. The systems listed in Table 2.1 will be presented in detail in the following sections.

<table>
<thead>
<tr>
<th>Classification system</th>
<th>Concept</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terzaghi, 1946</td>
<td>Rock load concept</td>
</tr>
<tr>
<td>Lauffer, 1958</td>
<td>Stand - up time</td>
</tr>
<tr>
<td>Deere, 1964</td>
<td>R Q D</td>
</tr>
<tr>
<td>Wickham et al, 1974</td>
<td>RSR System</td>
</tr>
<tr>
<td>Barton et al, 1974</td>
<td>Q System</td>
</tr>
<tr>
<td>Bieniawski, 1976</td>
<td>RMR System</td>
</tr>
<tr>
<td>Laubscher et al, 1976</td>
<td>Modified RMR system</td>
</tr>
</tbody>
</table>

2.2.3 Terzaghi's rock mass classification method

Terzaghi (1946) introduced the first method of rock classification. This method was applied to the support of tunnels which utilised steel sets. It was very simple and practical, but too general and did not provide any quantitative information on the properties of rock masses. Terzaghi's classification is based on his experiences in railroad tunnelling. One significant point in his work was that he tried to document his experiences so that they could be used by others in the design of tunnel support. Because
this method was introduced for estimating rock loads on steel set supports, it is not suitable for use in modern tunnelling where rock bolts and shotcrete are used.

2.2.4 Classification of Lauffer et al

Lauffer (1958) introduced the concept of the stand-up time of the unsupported active span of tunnels and related this to different classes of rock mass. An important point of the Lauffer system was that for a given rock mass quality any increase in tunnel span leads to a major reduction in stand-up time. The Lauffer classification originated from the earlier work of Stini (1950) on tunnel geology. Lauffer was the first to emphasise the importance of the stand-up time of the active span in a tunnel. The stand-up time is the period of time that a tunnel will stand unsupported after excavation. An active unsupported span is the width of the tunnel or the distance between the face and tunnel support, whichever is greater.

2.2.5 Deere's Rock Quality Designation (RQD)

Rock Quality Designation (RQD) was proposed by Deere in (1964) and since then it has been used as a major factor in rock mass classification methods such as the Q and RMR systems. RQD is defined as the ratio of the sum of sound pieces in a borehole sample greater than 100 mm length divided by the length of borehole. This produces the following:

$$RQD = 100 \times \frac{\text{Length of core in pieces > 100 mm}}{\text{Length of borehole}}$$

Determination of RQD is relatively quick and inexpensive, but the direction of a particular joint set is not considered and where the joint is filled with clay or weathered material it cannot be satisfactorily applied. Deere's method of classifying rocks is described as follows:

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

Table 2.2 Deere's rock mass classification based on RQD
2.2.6 Wickham et al RSR classification

Wickham, Tiedemann and Skinner (1974) introduced the method of Rock Structure Rating (RSR), the first complete rock mass classification system developed since Terzaghi's classification. This method was developed in the USA and it allows the quality of rock structure associated with ground support for tunnelling to be determined. One of the most important aspects of this classification was use of the concept of rating of a number of individual parameters for assessment of total rock mass quality. This means that different parameters are evaluated separately and the sum is used in the classification system. According to Bieniawski (1984) the RSR concept was a step forward in a number of ways; firstly, it was a quantitative classification, unlike Terzaghi's qualitative one; secondly, it was a rock mass classification incorporating many parameters, unlike the RQD index that is limited to core quality; thirdly, it was a complete classification having an input and an output, unlike a Lauffer-type classification that relies on practical experience to decide on a rock mass class, but then gives an output in terms of the stand-up time and span. As shown in Tables 2.3 to 2.5, there are three parameters A, B and C in this classification system and the resulting range of RSR values is between 19 and 107 and is given by the sum of A, B and C. The higher the number the better the rock, and vice versa.

**Table 2.3** Rock structure rating - Parameter, A: general area geology

<table>
<thead>
<tr>
<th>Basic rock type</th>
<th>Geological structure</th>
<th>Massive</th>
<th>Slightly faulted or folded</th>
<th>Moderately faulted or folded</th>
<th>Intensely faulted or folded</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Hard</td>
<td>Medium</td>
<td>Soft</td>
<td>Decomp</td>
</tr>
<tr>
<td>Intensely igneous</td>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Metamorphic</td>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Sedimentary</td>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Type 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>30</td>
</tr>
<tr>
<td>Type 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>27</td>
</tr>
<tr>
<td>Type 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td>Type 4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>19</td>
</tr>
</tbody>
</table>
Table 2.4 Rock structure rating - Parameter B: joint pattern, direction of drive

<table>
<thead>
<tr>
<th>Average spacing</th>
<th>Strike (\perp) to axis</th>
<th>Strike (\parallel) to axis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Direction of drive</td>
<td>Direction of drive</td>
</tr>
<tr>
<td></td>
<td>Both</td>
<td>With dip</td>
</tr>
<tr>
<td>Dip of predominant joints*</td>
<td>Flat</td>
<td>Dipping</td>
</tr>
<tr>
<td>1. Very closely jointed &lt; 110 mm</td>
<td>9</td>
<td>11</td>
</tr>
<tr>
<td>2. Closely jointed, 110 - 320 mm</td>
<td>13</td>
<td>16</td>
</tr>
<tr>
<td>3. Moderately jointed, 320 - 640 mm</td>
<td>23</td>
<td>24</td>
</tr>
<tr>
<td>4. Moderate to blocky, 0.6 - 1.2 m</td>
<td>30</td>
<td>32</td>
</tr>
<tr>
<td>5. Blocky to massive, 1.2 - 2.4 m</td>
<td>36</td>
<td>38</td>
</tr>
<tr>
<td>6. Massive, &gt; 2.4 m</td>
<td>40</td>
<td>43</td>
</tr>
</tbody>
</table>

*Joint condition; Good = tight or cemented; Fair = slightly weathered or altered; Poor = severely weathered, altered or open, lps = litres per second.

Table 2.5 Rock Structure Rating - Parameter C: ground water, joint condition

<table>
<thead>
<tr>
<th>Anticipated water inflow (lps/100 m)</th>
<th>Sum of parameters A+B 13-44</th>
<th>Sum of parameters A+B 45-75</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint condition*</td>
<td>Sum of parameters A+B 13-44</td>
<td>Sum of parameters A+B 45-75</td>
</tr>
<tr>
<td>Good</td>
<td>22</td>
<td>25</td>
</tr>
<tr>
<td>Fair</td>
<td>18</td>
<td>22</td>
</tr>
<tr>
<td>Poor</td>
<td>12</td>
<td>18</td>
</tr>
<tr>
<td>None</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slight</td>
<td>19</td>
<td>23</td>
</tr>
<tr>
<td>&lt; 4 lps</td>
<td>15</td>
<td>16</td>
</tr>
<tr>
<td>Moderate, 4 - 10 lps</td>
<td>15</td>
<td>16</td>
</tr>
<tr>
<td>Heavy &gt; 10 lps</td>
<td>10</td>
<td>14</td>
</tr>
</tbody>
</table>

*Joint condition; Good = tight or cemented; Fair = slightly weathered or altered; Poor = severely weathered, altered or open, lps = litres per second.
Wickham et al prepared a series of charts for determining typical ground support systems based on the RSR method. Charts for 3 m, 6 m, 7 m and 10 m diameter tunnels are available. An example is given in Figure 2.1 for a 6 m diameter tunnel.

![Support Chart for 6 m Diameter Tunnel](image)

**Figure 2.1** RSR concept: support chart for a 6 m diameter tunnel (after Wickham et al 1974).

The three steel rib curves indicated in Figure 2.1 reflect typical sizes used for the particular tunnel size. The curves for rockbolts and shotcrete are dashed to emphasise that they are based on assumptions and were not derived from case histories. The charts are applicable to circular or horseshoe shaped tunnels with similar widths. According to Bieniawski (1984) the RSR method is suitable for selecting steel rib support and it is not recommended for design of rockbolts and shotcrete support because it is an empirical approach and should not be applied beyond its range without sufficient and reliable data being obtained.

### 2.2.7 Bieniawski's geomechanics classification

Bieniawski (1976) developed a geomechanics based classification or Rock Mass Rating (RMR) system. The system operates by summing the values of five parameters, resulting in five different classes of rock being distinguished. These parameters are:

1) Rock material strength ($\sigma_C$)
2) Rock quality designation (RQD)
3) Spacing of joints
4) Joint condition
5) Ground water condition

In Table 2.6, the method of incorporating these parameters for the classification of rock masses is shown. According to this table the RMR values are between zero and one hundred. In this classification the higher numbers belong to very good rocks and the low numbers poor rocks. Since in this classification the orientation of the strike and dip of discontinuities has not been considered, Table 2.7 which is based on the studies of Wikham et al (1974) should be incorporated in the analysis.

**Table 2.6 Geomechanics classification of rocks (after Bieniawski, 1984)**

<table>
<thead>
<tr>
<th></th>
<th>Uniaxial compressive strength</th>
<th>&gt;250 MPa</th>
<th>100-250 MPa</th>
<th>50-100 MPa</th>
<th>25-50 MPa</th>
<th>5-25</th>
<th>1-5</th>
<th>&lt;1</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Rating</td>
<td>15</td>
<td>12</td>
<td>7</td>
<td>4</td>
<td>2</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>ROD</td>
<td>90% - 100%</td>
<td>75% - 90%</td>
<td>50% - 75%</td>
<td>25% - 50%</td>
<td>&lt;25%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Rating</td>
<td>20</td>
<td>17</td>
<td>13</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Joint spacing</td>
<td>&gt;2 m</td>
<td>0.6 - 2 m</td>
<td>200 - 600 mm</td>
<td>60 - 200 mm</td>
<td>&lt;60 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Rating</td>
<td>20</td>
<td>15</td>
<td>10</td>
<td>8</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Joint condition</td>
<td>Very rough surface not continuous no separation hard joint wall rock</td>
<td>slightly rough surfaces separation &lt;1 mm, hard joint wall rock</td>
<td>Slightly rough surfaces, separation &lt;1 mm, soft rock wall joint</td>
<td>Slickenslided surfaces or gouge &lt;5 mm, thick or open joint 1-2 mm, continuous joints</td>
<td>Soft gouge &gt;5 mm, thick or open joint &gt;5 mm, continuous joints</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Rating</td>
<td>25</td>
<td>20</td>
<td>12</td>
<td>6</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Ground water, inflow per 10 m tunnel length</td>
<td>none</td>
<td>none</td>
<td>&lt;25 L/min</td>
<td>25 - 125 L/min</td>
<td>&gt;125 L/min</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Ground water, (joint water pressure) / (major principal stress)</td>
<td>0</td>
<td>0</td>
<td>0.0 - 0.2</td>
<td>0.2 - 0.5</td>
<td>&gt; 0.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Ground water</td>
<td>dry</td>
<td>Damp</td>
<td>Wet</td>
<td>Dripping</td>
<td>Flowing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Rating</td>
<td>15</td>
<td>10</td>
<td>7</td>
<td>4</td>
<td>0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 2.7 Effect of discontinuity strike and dip orientations in tunnelling.

<table>
<thead>
<tr>
<th>Strike perpendicular to tunnel axis</th>
<th>Drive with dip</th>
<th>Drive against dip</th>
<th>Drive with dip</th>
<th>Drive against dip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dip 45°- 90°</td>
<td>Dip 20°- 45°</td>
<td>Dip 45°- 90°</td>
<td>Dip 20°- 45°</td>
<td></td>
</tr>
<tr>
<td>Very favourable</td>
<td>Favourable</td>
<td>Fair</td>
<td>Unfavourable</td>
<td></td>
</tr>
<tr>
<td>Strike parallel to axis</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dip 20°- 45°</td>
<td>Dip 45°- 90°</td>
<td>Dip 0°- 20°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fair</td>
<td>Very unfavourable</td>
<td>Fair</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

After a final rating of the rock mass has been determined, reference should be made to Figure 2.2 to determine the stand-up time of an unsupported span and the maximum active span for different RMR values.

After classifying a rock mass according to Bieniawski’s RMR classification, each rock mass class can be related to a specific engineering problem. In the case of tunnels and underground excavations the output is the stand-up time and the maximum stable rock span for a given rock mass rating (Figure 2.2).

Figure 2.2 Relationship between the stand-up time of unsupported underground excavation spans and RMR system (after Bieniawski 1976).
By using the RMR classification, Unal (1983) determined the support load and showed the variation of rock load as a function of roof span in different rock classes (Figure 2.3).

![Figure 2.3 Variation of rock-load as a function of roof span in different rock classes in the Geomechanics Classification (after Unal, 1983).](image)

The relationship between rock load, span and rock mass rating shown in Figure 2.3 was derived from the following equations:

\[ P = (100 - \text{RMR}) \times (\rho.B/100) = \rho.h \]  \hspace{1cm} \text{Eq. (2.1)}
h = \left[ \frac{100 - \text{RMR}}{100} \right] \times B \quad \text{Eq. (2.2)}

Where:

P = \text{Rock load, KN per unit length of tunnel}
B = \text{Tunnel width, m}
\text{RMR} = \text{Rock Mass Rating}
\rho = \text{Density of rock, kg/m}^3
h = \text{Rock load height, m}

Furthermore, based on his Geomechanics Classification, Bieniawski (1976) proposed a guide for the choice of support method of underground excavations. It should be noted that these support recommendations were for civil engineering tunnels of approximately 10 metres span excavated by the drill and blast method at depths of less than 1000 metres.

### 2.2.8 Q-System

Barton, Lien and Lunde (1974) from the Norwegian Geotechnical Institute (NGI) developed the Tunnelling Quality Index or Q-System. They studied many case histories of underground excavations and the results of their work was an index for the classification of rocks in which the numerical value, Q, was defined as follows:

\[ Q = \frac{\text{RQD} \cdot J_R \cdot J_W}{(J_n \cdot J_a \cdot \text{SRF})} \quad \text{Eq. (2.3)} \]

Where:

\text{RQD} = \text{Rock Quality Designation}
J_n = \text{Joint set number}
J_R = \text{Joint roughness number}
J_a = \text{Joint alteration number}
J_W = \text{Joint water reduction number}
\text{SRF} = \text{Stress reduction factor}

The numerical values of Q vary from 0.001 for very poor rock to 1000 for excellent rock. The above six parameters can be combined to give the following:

\[ \frac{\text{RQD}}{J_n} = \text{Block size} \]
\[ \frac{J_R}{J_a} = \text{Inter-block shear strength of joints} \]
\[ \frac{J_W}{\text{SRF}} = \text{Active stress} \]
The range of numerical values for the various parameters have been published elsewhere (Barton et al, 1976). It can be seen that the Q-System does not take the strength of the rock material or the direction of joints into account. As explained by Barton et al (1976) the parameters \( J_n, J_r \) and \( J_a \) are more important and if joint orientation had been included in this classification it would become less general. To relate the behaviour of an underground construction to the value of Tunnelling Quality Index (Q), and to determine the support required, the equivalent dimension \( D_e \), has to be defined. This is determined by applying the following equation

\[
D_e = \frac{\text{Excavation span, diameter or height, m}}{\text{ESR}}
\]

ESR stands for Excavation Support Ratio and is related to the underground excavation activities. Suggested values for ESR are given in Table 2.8:

<table>
<thead>
<tr>
<th>Excavation category</th>
<th>ESR</th>
</tr>
</thead>
<tbody>
<tr>
<td>A Temporary mine openings</td>
<td>3-5</td>
</tr>
<tr>
<td>B Vertical shafts:</td>
<td></td>
</tr>
<tr>
<td>- Circular section</td>
<td>2.5</td>
</tr>
<tr>
<td>- Rectangular/square section</td>
<td>2</td>
</tr>
<tr>
<td>C Permanent mine opening, water tunnel for hydroplant (excluding high-pressure penstocks), pilot tunnels, drifts and headings for large excavations.</td>
<td>1.6</td>
</tr>
<tr>
<td>D Storage rooms, water treatment plants, minor highway and railroad tunnels, surge chambers, access tunnels.</td>
<td>1.3</td>
</tr>
<tr>
<td>E Power stations, major highway or railroad tunnels, civil defence chambers, portals, intersections.</td>
<td>1</td>
</tr>
<tr>
<td>F Underground nuclear power stations, railroad stations, factories.</td>
<td>0.8</td>
</tr>
</tbody>
</table>

In Figure 2.4 the recommended maximum unsupported excavation spans for different Excavation Support Ratios (ESR) have been plotted against Q values. ESR values greater than 1.6 apply to temporary openings, and if spans greater than the design limits in this figure are excavated they may need some kind of support. A number of tables which relate the support required to the Q index have also been published (Barton et al, 1976). The following equation defines the actual span limits for permanently
unsupported openings. According to Barton this equation will always provide a conservative estimate of unsupported excavation spans.

\[
\text{Span} = 2 \cdot \text{ESR} \cdot Q^{0.4} \quad \text{Eq. (2.4)}
\]

From the above equation the Q value required for a particular span can be determined as follows:

\[
Q = (\text{Span} \cdot 2 \cdot \text{ESR})^{2.5} \quad \text{Eq. (2.5)}
\]

During the mapping of a tunnel or other underground excavation, the section requiring support can be readily identified from the above equation.

\[\text{Figure 2.4} \quad \text{Relationship between Equivalent dimension, } \text{De}, \text{ of an unsupported excavation and the tunnelling quality index, } Q \text{ (after Barton et al, 1976).}\]

It should be noted that the ESR values shown in Table 2.8 are only guide lines. Lower ESR values may be used in situations where there is a considerable doubt about the reliability of data used for obtaining the Q value. In order to estimate support requirements for the walls of a large excavation the wall dimension must be changed to an equivalent roof (span) dimension and the following modification to the Q values should be applied:

\[
\text{for } Q > 10 \quad Q_{\text{wall}} = 5 \cdot Q \quad \text{Eq. (2.6)}
\]
for $0.1 < Q < 10$ \[ Q_{\text{wall}} = 2.5 \, Q \] Eq. (2.7)

for $Q < 0.1$ \[ Q_{\text{wall}} = Q \] Eq. (2.8)

The dominant disturbing force is gravity and as such a wall will usually be more stable than a roof. The following support design chart, Figure 2.5, has been simplified from the original data (Stacey, 1986) and is appropriate for the design of primary support of civil, and for permanent support of mining excavations. For long term civil excavations the design chart output should be modified by halving the area supported per bolt:

![Figure 2.5](image)

Note: Where the area per bolt is greater than 6 m$^2$, spot bolting is implied).

**Figure 2.5** Bolt support estimation using the Q system (1,2,3...,9,10 means square metres of area of excavation surface per bolt)

The length of rockbolts or cable bolts can be calculated from the following formulae:

**For roofs:**

\[
L = 2 + 0.15B / \text{ESR} \quad \text{(bolts)} \]  
Eq. (2.9)

\[
L = 0.4B / \text{ESR} \quad \text{(cables)} \]  
Eq. (2.10)

**For walls:**

\[
L = 2 + 0.15H / \text{ESR} \quad \text{(bolts)} \]  
Eq. (2.11)

\[
L = 0.35H/\text{ESR} \quad \text{(cables)} \]  
Eq. (2.12)
Where:

\[
ESR = \text{Excavation Support Ratio} \\
B = \text{Span, (metres)} \\
H = \text{Wall height, (metres)} \\
L = \text{Rockbolt or cable bolt length, (metres)}
\]

### 2.2.9 Laubscher's Modified Rock Mass Rating System

The Modified Rock Mass Rating system was developed by Laubscher and Taylor (1976) and was particularly for mining situations. This system uses the same basic parameters as Bieniawski's system but has been modified in detail by taking the ground water and joint condition as one parameter and introducing the concept of insitu Rock Mass Strength (RMS) which relates the intact rock strength to the the local stress conditions. The parameters used are:

- RQD
- Intact Rock Strength (I.R.S.)
- Joint spacing
- Condition of joints and ground water

The appropriate rating for RQD, intact rock strength, joint spacing and joint condition and ground water are evaluated from various tables (Laubscher et al, 1977). The total rating value (A) is obtained by summing the above four parameters. The total rating values are between zero for very poor rock to 100 for very good rock. Using the above parameters a value for Rock Mass Strength can be determined from the following:

\[
\text{RMS} = C \times \left( \frac{A - B}{80} \right) \times 0.8
\]

Where:

- \(\text{RMS}\) = Rock Mass Strength, (MPa)
- \(A\) = Total Rating
- \(B\) = Intact Rock Strength rating
- \(C\) = Intact Rock Strength, (MPa)

Finally before applying the RMS results to design, some additional adjustments to take account of weathering, the influence of strike and dip orientations and blasting affects are considered as follows:
Weathering: Three parameters which can be affected by weathering are:

- RQD: an adjustment up to 95% being possible for RQD.
- I.R.S: an adjustment up to 96% being possible for Intact Rock Strength.
- Condition of joints: an adjustment to 82% being possible for this parameter.

Therefore, a total adjustment to 75% is possible for weathering effects.

Influence of strike and dip orientations: In the case where the direction of an underground opening is not oriented favourably with the strike and dip of discontinuities in a rock mass, an adjustment must be applied. This adjustment is specially important when the dimensions of the excavation are such that the blocks of the rock mass will be exposed. The percentage adjustment is shown in Table 2.9:

<table>
<thead>
<tr>
<th>Number of joints</th>
<th>Number of block faces inclined away from vertical and percentage adjustment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>70%</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
</tr>
</tbody>
</table>

Blasting effects: Blasting will create new fractures and cause damage in the surrounding rock mass, therefore, the percentage adjustment should be chosen based on the technique of blasting as shown in Table 2.10:

<table>
<thead>
<tr>
<th>Technique of blasting</th>
<th>Adjustment %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boring</td>
<td>100</td>
</tr>
<tr>
<td>Smooth wall blasting</td>
<td>97</td>
</tr>
<tr>
<td>Good conventional blasting</td>
<td>94</td>
</tr>
<tr>
<td>Poor conventional blasting</td>
<td>80</td>
</tr>
</tbody>
</table>

Mining stress environment: Mining-induced stresses are the redistribution of the field stresses after excavation. The maximum stress \( (\sigma_1) \), minimum stress \( (\sigma_3) \) and stress difference \( (\sigma_1 - \sigma_3) \) are the most important of these stresses. The redistribution of field
stresses can be obtained from finite element or boundary element analysis or from published stress redistribution data (Hoek & Brown, 1981).

After these adjustment the system proposes a Design Rock Mass Strength (DRMS) for a given rock mass as follows:

$$DRMS = RMS \times (Total\ adjustment)$$

Where:

- **DRMS** = Design Rock Mass Strength
- **Total adjustment** = (Weathering adjustment) × (Influence of strike and dip orientation adjustment) × (Blasting adjustment)

Knowing the value of DRMS and using the charts shown in Figure 2.6, the support required for an underground excavation can be selected. Each chart is divided into five zones as follows:

- **Zone I** is a stable zone and no support is needed.

- **Zone II** potentially has unstable blocks and support is needed. In certain cases the whole drift may need support particularly in near surface excavations. In this case joint condition, weathering and joint orientation should be considered.

- **Zone III** is a zone in which spalling, rock falls, movement on joints and plastic deformation occurs with increasing intensity as the mining environment stress increases.

- **Zone IV** represents areas in which significant spalling and shear movement on joint are expected. Total rock reinforcement is required and special attention must be paid to repair techniques. Ore extraction must be at the maximum rate as support will not provide permanent access.

- **Zone V** regular collapses and caving will occur, and support will not be successful.

When using the above charts the following criteria must be taken into consideration:

- Maximum induced stress which causes failure where the Design Rock Mass Strength (DRMS) is less than the mining environment stress.

- Minimum induced stress which leads to opening of joints, reduction of confinement and joint shear strength and ultimately falling out of blocks.
Figure 2.6 Support selection charts in which DRMS is related to: (a) maximum stress, (b) minimum stress, (c) difference between maximum and minimum compressive stresses and (d) support techniques and symbols (after Laubscher and Taylor, 1976)
2.2.10 Use of stability graph method to predict the safe span of an opening

The stability graph method (Mathews et al, 1981) was developed for open stope design and has been used successfully for the design of new stopes in a working mine or for the design of stopes in a new project. To use this method sufficient geotechnical data should be available. The following procedure should be followed for this design method:

a) At first the modified Q index \( Q' \) is obtained by:

\[
Q' = \frac{RQD}{J_I} \times \left( \frac{J_r}{J_n} \right) \times J_w
\]

Eq. (2.14)

In the modified Q index the Stress Reduction Factor (SRF) is assigned the value of 1.0.

b) Rock stress factor \( A \): This stress factor is determined using the ratio of the unconfined uniaxial compressive strength and the induced stress acting parallel to the exposed surface under analysis (Figure 2.7). Induced stresses are obtained from the particular charts which has been published elsewhere (Mathews et al, 1981).

\[ a = \text{Uniaxial compressive strength of intact rock} \]
\[ a = \text{Induced compressive stress} \]

**Figure 2.7** Graph to determine the rock stress factor 'A' (after Mathews et al, 1981)

(c) Rock Defect Orientation Factor \( B \): Usually this factor is determined with the help of a stereonet. The smallest acute angle of all joints, which is the result of the intersection of the most persistent joint set with the surface under analysis, is given highest priority (Figure 2.8).
d) The Design Surface Factor (C): This factor was introduced to explain that under the same rock conditions the backs of the stopes are less stable than hangingwalls, and hangingwalls are less stable than footwalls. For backs and hangingwalls the Factor 'C' can be obtained from the following formula or Figure 2.9.

\[ \text{Factor C} = 8 - 7 \cos(\text{angle of dip of the surface under study from horizontal}) \]
Figure 2.9 Factor C for backs and hangingwalls (after Mathews et al, 1981)

e) The Stability Number (N)

A stability number, 'N', is determined from the following equation:

\[ N = Q' \times A \times B \times C \]  

Eq. (2.15)

In the determination of the stability number the influence of rock mass quality, induced stresses, rock defects and the dip of the exposed surface have been considered.

Using the stability number the maximum allowable value for a shape factor, 'S', can be determined from a stability graph (Figure 2.10). The shape factor, 'S', is a function of the hydraulic ratio of a surface and as such, if one dimension of a surface is defined the maximum value for the other dimension is found using the maximum shape factor. Although \( S_{\text{max}} \) is determined from the stability graph, it can be modified using the judgment of the mining engineer (usually the result of previous experience). A critical path for stope design using the above method was suggested by Bawden et al (1989) and is shown in Figure 2.11.
2.2.11 Comparison of rock mass classification systems

In general rock mass classification systems are a good guide for designers and are useful when solving ground control problems. The approach may not provide a complete solution for engineering problems, but it leads to better engineering judgement. If there was a standard and common rock mass classification system it would be ideal, but for the time being because there are many different systems of classification it is better to try two or more systems for any engineering project to obtain improved understanding of the rock situation. It is important to know that empirical methods are not the only way of classifying rock masses and if possible analytical methods and physical modelling should be used. Furthermore each classification system gives reliable results only for the rock mass and circumstances for which it was developed, therefore, it shouldn't be applied to all the projects blindly. If a specific classification system is applied to a new project which is similar to the conditions for which the system was developed, then the results will be more reliable.

Barton, (1976) compared the Q-system with the geomechanics classification system and pointed out that the stress conditions considered in the Q system are not considered in the RMR system. On the other hand spacing and orientation of joints which are considered in the geomechanics classification have been omitted from the Q system. Bieniawski (1976) analysed a total of 117 case histories and obtained a correlation between Q and RMR as follows.
Figure 2.11 Critical path for stope design (after Bawden et al, 1989)
RMR = 9 Ln Q + 44 \hspace{1cm} \text{Eq. (2.16)}

Houghton (1976), compared the RSR, RMR and Q systems and pointed out that the difference between the proposed scale of support according to each method of classification reflects the different parameters used by each author, rather than an interpretation of the behaviour of the rock. Furthermore, when compared with Barton's Q system, the geomechanics classification is easier to apply in the field.

Generally rock mass classifications can be used as a first estimation of stable unsupported spans by using charts such as that shown in Figure 2.12. This chart shows the limits of unsupported span proposed by different systems. When defining the safe span for a particular crown pillar from these charts it should be noted that the geometry and geology used to develop these curves was different, and as such the curves should be used with caution.

Figure 2.12 Suggested limiting span prediction curves for Q and RMR systems (after Carter, 1989).
2.3 Analytical methods

2.3.1 Roof beams and plates

It may be assumed that a mine roof acts as a beam or plate supported by elastic abutments and has a total applied load equal to its self weight plus any load induced by the overlying strata. Actually an intact mine roof is a slab or plate because it is fixed at all edges. For most mining situations the long dimension is significantly greater than the short span and as such, beam and plate theories produce approximately the same values for maximum tensile strength. For shallow mines the roof beam is considered to be simply supported whilst for deep mines it may be considered to have fixed ends.

Accepted beam theory states that failure will occur when the maximum tensile stress is greater than the modulus of rupture of the rock. For a brittle material such as rock the tensile strength in flexure is greater than the strength in pure tension (Woodruff, 1966). However, in reality most mine roofs are jointed and simple beam theory is not valid.

2.3.2 Bending of beams and plates

A mine roof which is long in comparison with its width, with span to thickness ratio greater than two, is considered as a uniformly loaded beam fixed at both ends. The stresses and deflection can be estimated from beam or plate theory. When applying these theories the following assumptions are considered (Adler and Sun 1968):

- The strata is homogeneous, isotropic and behaves elastically.
- The thickness of the layer is small in comparison to the roof span.
- The length of the roof slab is more than twice its span for beams and less than twice for plates.

A slab is assumed to work as a beam or plate which is supported by elastic abutments and supports a uniformly distributed load which is equal to its weight, plus any load that can be induced by the overlying strata. For shallow mines the mine roof is considered to be a simple beam. For deep mines it is assumed that the mine roof is a beam with fixed ends. The maximum deflection, shear and tensile (or compressive) stresses on a simply supported uniformly loaded beams are found from the following formulae (Obert and Duvall, 1967):
\[ \delta_{\text{max}} = \frac{\gamma L^4}{32Et^2} \quad \text{Eq. (2.17)} \]

\[ \tau_{\text{max}} = \frac{3 \gamma L}{4} \quad \text{Eq. (2.18)} \]

\[ \sigma_{\text{max}} = \frac{\gamma L^2}{2t} \quad \text{Eq. (2.19)} \]

where:

- \( L \) = Roof span, m
- \( t \) = Roof thickness, m
- \( E \) = Young's modulus, GPa
- \( \gamma \) = Weight density of the rock, MN / m³
- \( \delta_{\text{max}} \) = Maximum deflection, mm
- \( \tau_{\text{max}} \) = Maximum shear stress, Mpa
- \( \sigma_{\text{max}} \) = Maximum tensile stress, MPa

### 2.3.3 Calculation of mine roof safe span

(a) Simply supported roof beams

The maximum moment for a simply supported beam with uniform load is as follows (Corlett & Emery, 1959):

\[ M = \frac{WL^2}{8} \quad \text{Eq. (2.20)} \]

Where:

- \( M \) = Maximum moment in the beam, N m
- \( W \) = Load on each unit length of beam, N/m
- \( L \) = Span of the beam, m

The section modulus for a beam with rectangular cross-section is:

\[ I/C = b \frac{d^2}{6} \quad \text{Eq. (2.21)} \]

Where:

- \( I \) = Second moment of roof beam section (moment of inertia), m⁴
C = Distance from the centre of gravity to the extreme fibre, m
b = Width of the roof beam, m
d = Roof beam thickness, m

The maximum stress developed in the top and bottom fibres is:

\[ \sigma_m = \pm \left( \frac{M C}{I} \right). \]

If \( \sigma_x \) is the resultant stress due to prestress in the extreme fibre, the uniform load which should be applied to prevent deflection and tension is obtained as follows:

\[ W = \frac{8 \times I \times \sigma_x}{C L^2} \quad \text{Eq. (2.22)} \]

Considering Equation 2.21 and 2.22 when the width of the beam is equal to unit width the safe span can be calculated using the following Equation:

\[ L = 2 \sqrt{\frac{d.\sigma_x}{3.\gamma F}} \quad \text{Eq. (2.23)} \]

Where:

L = Safe span, (m)
d = Thickness of the beam, (m)
\( \sigma_x \) = Resultant stress due to prestress, (MPa)
F = Factor of safety usually between 4 to 8
\( \gamma \) = Weight density of the rock, MN / m³

(b) Fixed end roof beams

For a mine roof span greater than twice its thickness, the roof is treated as a uniformly loaded beam fixed at both ends. Because rock is much weaker in tension than in compression, in this case only tensile and shear stresses are considered critical (Adler and Sun 1968). The tensile stresses developed by the beam are significantly higher than the shear stresses. The maximum deflection occurs at the centre of the beam and maximum tensile stress occurs in the top surface of the slab near the abutments. The compressive stress in the lower part of the beam at the abutments is not considered because rock is much weaker in tension than in compression or shear. Failure will be initiated on the top surface at the ends of the span, because shear and tensile stresses are maximum at this point.
\[ \sigma_{\text{max}} = \frac{W S^2}{2d} \quad \text{Eq. (2.24)} \]
\[ \tau_{\text{max}} = \frac{3W S}{4} \quad \text{Eq. (2.25)} \]

where:
- \( \tau_{\text{max}} \) = Maximum shear stress, MPa
- \( \sigma_{\text{max}} \) = Maximum tensile stress, MPa
- \( S \) = Span of beam, m
- \( d \) = Thickness of beam, m
- \( W \) = Roof beam total load, MN/m³

The ratio of \( \sigma_{\text{max}} \) to \( \tau_{\text{max}} \) is:

\[ \frac{\sigma_{\text{max}}}{\tau_{\text{max}}} = \frac{2S}{3d} \quad \text{Eq. (2.26)} \]

For a span to thickness ratio greater than 5 to 1, the tensile stress is more than three times the shear stress. The tensile strength of the rock is usually less than the shear strength and much less than the compressive strength, therefore, the design of a safe span is based only on the maximum tensile stress in the roof layer. The above Equation can be rewritten as a design formula for the determination of the critical span:

\[ S_{\text{max}} = \sqrt{\frac{2Rd}{\gamma}} \quad \text{Eq. (2.27)} \]

If 'S' is divided by a safety factor, the safe span of the roof can be given by:

\[ S = \sqrt{\frac{2Rd}{\gamma F}} \quad \text{Eq. (2.28)} \]

where:
- \( S \) = Safe roof span (m)
- \( R \) = Modulus of rupture of the roof rock (tensile strength in flexure) (MPa)
- \( d \) = Thickness of the roof beam (m)
- \( F \) = Factor of safety (from 4 to 8)

By using Equation 2.28 the relationship between safe span, tensile strength of the rock
and roof thickness can be demonstrated in a graph as shown in Figure 2.13 (Adler and Sun, 1968)

![Figure 2.13 Computed safe span versus roof thickness for different tensile strengths (after Adler and Sun, 1968)](image)

**Figure 2.13** Computed safe span versus roof thickness for different tensile strengths (after Adler and Sun, 1968)

### 2.3.4 Cracked roof beams

Self supporting spans of cracked roof beams are common in underground mines. In some cases they may be the roof of an open stope and in other cases it may be a crown pillar which stands between two open stopes.

The cracks may be natural, induced mining fractures or tension cracks developed at the bottom of a roof midspan and at the top of the beam at the abutments. Even if the rock in a roof beam fails in tension it does not mean that the beam will collapse. This suggest that roof beam design should not be based solely on beam theory and the tensile strength of the rock or its modulus of rupture.

The fact that a jointed slab can stand safely when the abutments are not free to move outward has long been recognised. Bucky (1934) conducted model studies to determine the effect of vertical and steeply inclined cracks in rock beams. The results from these studies showed that cracked rock beams can stand without artificial support, provided the ends of the beam are constrained. This result is practically independent of the number of
cracks. Evans (1941) conducted further model studies on cracked beams (voussoir beams) in which he measured the horizontal forces resulting from downward deflection of the beam causing a 'natural compressive arch' to be set up in the beam. The complete analysis by Evans assumed a no tensile situation and that the compressive stress at the mid-span and at the abutments has a triangular distribution. Further, he stated that an assumption of the vertical length of contact along the centre and abutment cracks must be made prior to analysis; and that a value of half the beam thickness was reasonable. This latter assumption was found to be in error by Wright (1972) who claimed that an iterative approach could be used to determine a more accurate value. This theory of the 'Voussoir Arch' is discussed in detail in chapter 3.

Mohr (1963) also studied models of cracked beams and made some most interesting observations as follows:

- A beam supports itself if there is a certain lateral force pressing the vertical points together.

- As soon as the lateral pressure decreases, the beam begins to bend. This bending ceases as soon as the horizontal force is increased again.

- A thin wooden beam can bend much further than a thick one before breaking. More complex are the results if one tries to prevent the bending through some type of support. The force necessary to do so is much lower with a thick beam than with a thin one.

Wright (1972) used experimental methods to investigate the effect of arching action in a cracked roof beam. In his work he showed that an arch is formed in a mine roof with vertical joints when the pre-existing lateral stresses are not great enough to preclude any horizontal tensile stress in the beam. Some formulae were derived for thrust, deflection and maximum stress in such beams. It was claimed that when the calculated deflection at the centre of the beam exceeds 14 percent of the depth of the beam buckling failure is possible.

An extensive series of tests were conducted on laterally constrained rock beams by Sterling (1980) with a number of conclusions being reached. For span/depth ratios above 20 the beam will fail by buckling, for ratios between 5 and 20 failure would be compressive, and for ratios less than 5 failure if it occurred would be by shear at the abutments. The actual ratio at change points would depend on the loading and support
conditions. The increase in ultimate vertical load capacity was greatest from zero pre-stress to a value of around 3-5 MPa (for the particular case analysed), but beyond this value the ultimate vertical load capacity was increased only slightly by further pre-stress.

Further work on the application of voussoir beam and plate theories by Beer and Meek (1982) concentrated on developing a series of design curves for roofs and hanging-walls in bedded rock formations. The parameters considered in the design were strata thickness, dip angle, elastic modulus and compressive strength whilst the basic assumption for solution was that the roof or hangingwall consisted of a series of no tension beams or plates. The design curves produced indicated safe or unsafe spans for a given strata thickness, but did not include the effect of the horizontal stress field existing in a mining situation. These curves are shown in Figure 2.14 and 2.15, and they assume plain strain conditions provide conservative estimates of the stable span for openings with a length to width ratio less than 3.

## 2.4 Numerical methods

Empirical and theoretical methods do not thoroughly analyse crown pillar behaviour. They can relate stress and dimensions but the variation in stress and displacement inside the pillar and adjacent rock cannot be described using these methods. Numerical modelling is able to provide the details of stress and displacement inside the crown pillar. Numerical analysis is divided into two methods: the continuum approach which considers the rock mass as a continuum block with a number of discontinuities, and the discontinuum approach which considers the rock mass as a group of independent blocks. Finite element and boundary element methods are different types of continuum method. Distinct element analysis is an example of the discontinuum method.

Any of the above methods will give a reasonable approximation for the elastic stress field around an opening if the limitations associated with each method are considered. There are two possible avenues for obtaining the stress field around an opening:

(a) Calculating the perturbation from the pre-mining stress field due to the opening and consequent addition to the original stress field.

(b) Applying total stresses at a distance from the boundary (or considering gravity loading) and calculating the total stress field in one operation.
Figure 2.14 Roof beam stability design curves (after Beer and Meek, 1982)
Figure 2.15 Roof beam stability design curves (after Beer and Meek, 1982)
The first method is better because it gives the displacement values that are measured in the field, but the second method can lead to unforeseen computational difficulties because of the presence of large displacements.

There are many different computer programs available for use in analysis of stress conditions around underground openings, however, only a few of them can evaluate complex geometrical and geological structures such as crown pillars. Elastic stress analysis even for complex geometries is relatively easy but for analysing post-failure behaviour of underground structures especially for crown pillars the available programs are limited. In many cases the results are very sensitive to input parameters of the model geometry.

Many computer programs which are based on a continuum mechanics formulation (ie. finite element and finite difference programs) can simulate the variability in material types and nonlinear constitutive behaviour typically associated with a rock mass, but modelling the discontinuities requires a discontinuum-based formulation (ITASCO Consulting Group, 1993). Some finite element and finite difference programs can model a discontinuous material to some extent, however, their formulation is usually restricted in one or more of the following ways. First, the logic may break down when many intersecting interfaces are used; second there may not be an automatic scheme for recognising new contacts; and, third, the formulation may be limited to small displacements and rotation. For these reasons continuum methods are restricted in their applicability for analysis of underground excavations in jointed rock.

Usually many significant strains and displacements occur before final collapse in crown pillars. At present there are two commercially available programs which have been developed specifically for rock and soil mechanics purposes. These two programs are FLAC (Fast Lagrangian Analysis of Continua) and UDEC (Universal Distinct Element Code). Both of them can provide an elastic solution as well as post-failure processes. UDEC can model large interblock movements and is suitable for modelling jointed rock masses.

When using numerical techniques for the analysis of stress and displacement, field parameters should be measured correctly. The basic information necessary for most types of analysis are:

(a) Zones of rock type and their elastic properties (E,v);
(b) The pre-mining stress field which can be calculated as per equations 1.1 and 1.2.
However, experience has shown that horizontal stresses are usually much higher than predicted by the above, possibly the result of the presence of significant tectonic forces, or other 'locked-in' stresses (Duvall, 1977). Virgin stress measurement is necessarily expensive as it must be made in some part of the mine remote from any significant stoping. However, values should at least be checked so that they satisfy equilibrium requirements and the vertical stress should equal the gravity depth stress.

### 2.4.1 Finite Element method

With the finite element method fractures and weakness zones in deep openings can be modelled by using a suitable failure criteria (Meek & Beer 1984). Also using this method irregular geometries, non-uniform materials, non-linear behaviour and different kinds of loading can be modelled. Input information for finite element analysis includes a mesh which reflects the size and shape of the area under study and the mechanical properties of each element in the mesh; usually density and Young modulus. Output from a finite element program is usually in the form of stresses and displacements within the mesh. These results can be used to help in the determination of potential zones of instability or, by using parametric analysis, propose a suitable size and shape for an excavation.

In its simplest form, the finite element method has restrictions for use in geomechanics applications because of the necessity to model the far field. Because displacements remote from the opening must be set to zero, a large portion of the finite element mesh is simply used to model the far field. This difficulty has been solved in two ways: (a) the development of the infinite element which includes in their formulation a decay term, or (b) the coupling of the boundary element and finite element. This second technique has been successfully applied to two dimensional problems (Meek & Beer 1984).

### 2.4.2 Boundary Element method

Recently, the boundary element method has been used for stress analysis in rock masses around underground excavations (Meek & Beer 1984). Most boundary element programs assume that the rock mass surrounding an opening is an isotropic and homogeneous medium and as such is suitable for modelling by linear homogeneous elastic systems. Boundary element methods can only provide stress and displacement along interior or exterior boundaries. This technique is economical for two and three dimensional rock mass analysis particularly when the boundaries are considered, and is used for rapid evaluation of stress around underground excavations. The method allows three options of insitu stress input:
(1) Gravitational vertical stress with a given ratio of horizontal to vertical stress, 'k'.

(2) Elevated horizontal stress in the near surface zones plus gravitational stress conditions with given 'k' at depth.

(3) Uniform horizontal and vertical stresses with a constant "k".

Case 1 and 2 generally represent the insitu stress conditions of most crown pillar situations, while case 3 can be used to represent insitu stress conditions at depth.

2.4.3 Finite Difference method

With finite difference, as in the finite element method, inhomogenity in the rock mass can be modelled better than by the boundary element method (Meek & Beer 1984). FLAC (Fast Lagrangian Analysis of Continua) is a finite difference method which can model large strains and therefore, is suitable for modelling the crown pillar situation. With explicit finite difference the distorted shape of the underground opening can be seen and this is useful for understanding the initiation and propagation of failure (Carter, 1989).

2.4.4 Distinct Element method

The distinct element method, beginning with initial presentation by Cundall (1971), has been formulated based on a discontinuum approach for simulation of jointed rock masses. In the distinct element method, a rock mass is represented as a group of discrete blocks which may be rigid or deformable. If a block is assumed to be rigid its geometry does not change as a result of applied forces. Consequently, the use of rigid blocks is only applicable to problems in which the behaviour of the system is dominated by discontinuities and for which the material elastic properties may be ignored. Such conditions arise in low-stress environments and/or where the material possesses high strength and low deformability. Joints are viewed as interface between distinct bodies (i.e., the discontinuity is treated as a boundary condition). The response to applied load in the distinct element method is based on a force-displacement law which specifies the interaction between the blocks which Newton's second law and determines displacements in the blocks. The contact forces and displacements at the interfaces of a stressed group of blocks are found through a series of calculations which trace the movement of the blocks (ITASCO Consulting Group, 1993).

Distinct element programs can represent the motion of multiple, intersecting
discontinuities explicitly. They use an explicit time-stepping scheme to solve the equation of motion. Bodies may be rigid or deformable and contacts are deformable. Cundall and Hart (1989) provide the following definition of a discrete element method (ITASCO Consulting Group, 1993).

(a) It allows finite displacement and rotation of discrete bodies, including complete detachment;

(b) It recognises new contacts automatically as the calculation progresses.

2.4.5 Computer program for modelling discontinuous systems

UDEC is the distinct element program which has been used in this study and allows the modelling of two different types of behaviour:

1. Point contact with elastic material and Mohr-Coulomb failure criterion.
2. Point area contact with elastic plastic material and Mohr-Coulomb failure criterion.

For many applications the deformation of individual blocks cannot be ignored (ie. blocks cannot be assumed to be rigid). Therefore, 'fully deformable' blocks were developed in UDEC to permit internal deformation of each block in the model. The program UDEC divides all of the blocks which have been created within the model into triangular finite difference zones. The basic failure model for blocks in UDEC is the Mohr-Coulomb failure criterion. Other non-linear plastic models recently added to the program include ubiquitous joint model and strain-softening models for both shear and collapse of blocks.

2.5 Design guidelines for the support of crown pillars

Today artificial support of stopes and crown pillars is a common practice in the mining industry. Use of cable bolts is relatively new in supporting stopes, and therefore, design is based on trial and error or past experience in most mines. This often results in overdesigned support or inadequate support which can lead to stope or crown failure. The original support philosophy was to suspend the loose rock around an excavation to more competent and undisturbed layers remote from the opening surface. Later a better understanding of rock mass behaviour and support systems led to the development of a better and more efficient technique called pre-reinforcement. The concept of pre-reinforcement is aimed at helping the rock mass to stand by itself by minimising the
disturbance of the rock mass.

Potvin et al (1989) used the stability graph method (Mathews et al, 1981) to develop a method for the determination of support required for open stopes. The factors they considered for the design of a cable bolt support system include bolt length, bolt density and the rock mass quality. Some empirical charts were also developed, based on case histories of cable bolting in Canadian open stope mines, which allowed comparison between cable bolt length and hydraulic radius, rock quality and a bolting factor and also between hydraulic radius and a stability number. These charts will be explained in detail.

Cable support systems should be applied according to the nature of the rock mass and the potential rock mass failure mechanism. The cable bolt patterns shown in Figures 2.16 and 2.17 are based on support systems used in Canadian mines and show a uniform distribution of cables. The length of cable associated with these patterns varies from 10 m to 25 m and the density of bolting is designed at 0.1 to 0.4 cable bolts per square metre. In some cases a set of 2 to 3 metre grouted reinforcing rock bolts are installed between the cables, at a density of 0.7 rock bolts per square metre. The objective of this work is to create a rock beam supported with the short rock bolts and tie the beam to more competent layers with the cable bolts.

The support system shown in Figure 2.16 is generally applied when the overcut is fully open. The second support system as shown in Figure 2.17 (a) tries to take advantages of the concept of prereinforcement. The overcut in this case is driven in two stages. The central section (c) is opened first and cable bolts are installed vertically in the back of the open section. Supplementary cable bolts are also installed at an angle, over the sides during the second stage of development. Another modification of open stope bolting is shown in Figure 2.17 (b). In this case cable bolts are installed at an inclination of 76 degrees in one direction for a given row while in the next row the cables are inclined in the opposite direction. This alternate inclined pattern aims at intersecting geological discontinuities at a more favourable angle.
Figure 2.16 Uniform cable bolt pattern installed in open stope; (a) without short rock bolts, (b) with short rock bolts.

Figure 2.17 Cable bolt support system: (a) With inclined cables and two phases of overcut development for prereinforcement, (b) With interlaced pattern.
A modified stability graph has been developed (Potvin et al, 1989) which can be used as a design guide for the support of open stopes. This stability graph shows the stable, unstable and caved zones and can be used as a guideline for design of open stopes (Figure 2.18).

![Figure 2.18 Revised stability graph (after Potvin et al, 1989)](image)

**Figure 2.18** Revised stability graph (after Potvin et al, 1989)

### 2.5.1 Prediction of support density

For better performance of a cable bolt system three principal variables should be considered. These are the density of bolting, the length of the cable bolts and their orientation. The orientation of cable bolts should be designed based on the predicted mode of failure. For gravity falls or slabbing the cable bolts should be installed vertically. In cases where the mode of failure is shear or sliding, the best design is to install the cables at an angle between 17 and 27 degrees to the shear direction (Miller, 1984). The density of the cable bolt support for stope backs can be determined from a method proposed by Potvin (1989). In this method the modified Q value ,Q', is adjusted for the relative intensity of support (Figure 2.19). Figure 2.19 should be used in areas with a uniform bolting pattern. The length of cable bolts can be obtained from Figure 2.20.
Figure 2.19 Adjustment of modified Barton classification ($Q'$) for the relative intensity of cable bolt support (after Potvin, 1988).

Figure 2.20 Design chart for the cable bolt length (after Potvin et al, 1989)
2.6 Conclusions

The crown pillar is a complex underground structure and the methods available for assessing its stability are not adequate. Until now only limited research has been done on this subject and in order to develop an appropriate design method further research is necessary.

Obtaining accurate properties of rock leads to a better understanding of the possible modes of pillar failure. Rock mass classification is a good guide for estimation of open spans in underground excavations, but at present no particular system is suitable for all conditions. It is better to try two or more systems in an engineering project as this will help to obtain a better understanding of rock behaviour. When developing a design method different modes of failure should be recognised. Back-analysis of past pillars is a particular way of identifying the potential mode of failure in a crown pillar. When it is used in a surface crown pillar failure back-analysis shows that horizontal stresses are low at the time of failure. Normally the stresses are higher in a crown pillar at depth when compared with surface crown pillar situations, therefore, the mode of failure may change from say shear at the surface to buckling or compressive failure at depth. Numerical methods can be used to study the stress regime surrounding and within a crown pillar.

At present there are only a few empirical methods available for the design of crown pillar support. Preparing a suitable model for the complicated ground conditions often associated with a crown pillar and obtaining suitable results from these methods for the design of a support system is very difficult. It can therefore be more appropriate to choose an empirical design and modify it to suit the local geological conditions. From the study described above the following points can be concluded:

(a) High or very low compressive stresses and structural geology appear to be the most important factors influencing the stability of crown pillars.

(b) The geometry of the crown pillars and mining practice plays a significant role in the stability condition and all of these factors can be used to determine the possible mode of failure of a crown pillar.
CHAPTER 3

EVALUATION OF PROCEDURE FOR THE DESIGN OF A CROWN PILLAR

3.1 Introduction

The stability of crown pillars of different configuration is difficult to assess because of variation in rock types, changes of geological structure, stress conditions, mining sequence and extraction ratios from one site to another. Although many procedures have been devised to help consider the interaction of all parameters when designing crown pillars, it is still a very complex problem which requires further study.

Recent investigations for design of crown pillars (Hunt, 1989) shows that in comparison with other design methods the empirical method that compares a stability factor with the hydraulic radius (Mathews et al, 1981) is more successful. This method can be applied where extensive geological, geometrical and rock mass data are available, however, when used for the design of crown pillars it cannot predict the thickness.

In this chapter a method of crown pillar design has been developed based on available empirical and theoretical formulae and design charts. This method enables the prediction of optimum thickness for a crown pillar. For this purpose voussoir beam and tributary area theories have been modified and based on the work of Potvin et al (1989) a design chart for support of crown pillar is suggested. The proposed method can be used both for crown pillar and surface crown pillar design.
3.2 Stress in pillars

The stress acting on a pillar depends on the pre-mining stress field, extraction ratio, location of the pillar, width to height ratio of the pillar and the physical properties of the rock. Of these factors, only the pre-mining stress field and the extraction ratio have a major effect on induced pillar stress (Hedley and Grant, 1972). The average stress on crown pillars can be approximated by the tributary area theory. It may be noted that this theory calculates the upper limit of the average pillar stress.

3.2.1 Tributary area theory

This theory expresses the pillar load for rectangular pillars as follows (Figure 3.1)

\[
\sigma_p = \frac{\sigma_v (w_o + w_p)(w_o + L)}{w_p \times L} \tag{3.1}
\]

Where:
- \( \sigma_p \) = Pillar load or the average pillar stress, MPa
- \( \sigma_v \) = Virgin vertical stress, MPa
- \( H \) = Depth below surface, m
- \( L \) = Pillar length, m
- \( B \) = Entry width, m
- \( W_p \) = Width of Pillar, m
- \( W_0 \) = Width of opening, m

The value of \( \sigma_p \) is obtained from the overburden weight above the seam. The pressure can generally be considered to increase at a rate of 0.025 MPa per metre of depth (Bieniawski, 1984) for coal formations.

The extraction ratio is obtained from the following formula (Figure 3.1):

\[
e = 1- \frac{((W_p \times L_p)/((W_0 + W_p) \times L_p)}
\]

or:

\[
e = 1-[W_p/(W_0 + W_p)]
\]

This formula can be rewritten as follows:

\[
e = 1- \left[ \frac{1}{((W_0/W_p) + 1)} \right] \tag{3.2}
\]
Figure 3.1 Tributary area theory and the way of expressing the pillar load for rectangular pillars (Brady and Brown, 1985)

Where:

\[ W_p = \text{Width of pillar, m} \]
\[ W_o = \text{Width of opening or stope height, m} \]
\[ L_p = \text{Length of pillar, m} \]

Therefore, Equation 3.1 can be rewritten as follows:

\[ \sigma_p = \frac{\sigma_y}{1 - e} \]

Eq. (3.3)
Where:

\[ \sigma_p = \text{Average pillar stress, MPa} \]
\[ \sigma_v = \text{Vertical stress, MPa} \]
\[ e = \text{Extraction ratio} \]

Using Equation 3.2 and 3.3 a graph relating the \( \text{Wo/Wp} \) ratio to the pillar stress/pre-mining stress can be obtained as shown in Figure 3.2.

Coates (1966) through his investigations found that the tributary area theory predicts the pillar stress 40 percent higher than the actual average value. During field measurements Hustrulid (1981) also found that the tributary area method estimation is 40 percent higher than the actual average stress on the pillar. Therefore, a reduction factor of 40% is inserted into Equation 3.3 and it is modified as follows:

\[ \sigma_p = 0.6 \sigma_v / (1 - e) \quad \text{Eq: (3.4)} \]

**Figure 3.2** Relation between \( \text{Wo/Wp} \) and pillar stress/pre-mining stress based on tributary area theory.

Using Equation 3.4, the graph in Figure 3.2 is modified as shown in Figure 3.3.
For orebodies which are inclined, the pre-mining stress $\sigma_0$, is a combination of the component of vertical stress $\sigma_v$ and horizontal stress $\sigma_h$ and can be obtained from this Equation:

$$\sigma_0 = \sigma_v \cos^2 \alpha + \sigma_h \sin^2 \alpha$$  \hspace{1cm} \text{Eq. (3.5)}$$

Where:

$\sigma_0$ = Pre-mining stress normal to orebody, MPa
$\alpha$ = Angle of orebody inclination, degrees

The vertical stress $\sigma_v$ is assumed to be due to the weight of the overlying strata, and increases at a rate of 0.025 MPa per metre.

$$\sigma_v = 0.025 H$$  \hspace{1cm} \text{Eq. (3.6)}$$

Where; $H$ = Depth from the surface, m

### 3.2.2 Stress estimation in the crown pillar

In general crown pillar stress can be obtained from the different numerical stress analysis methods. However, in this section tributary area theory has been modified to obtain crown
pillar stress as shown in Figure 3.4. It is assumed that after excavation of the stope the load which was carried by it is distributed onto the crown pillars and abutment pillar (or rib pillars). Figure 3.4 shows the crown pillar carries the load of parts 3 and 1 of the upper and lower stope. The loads of parts 2 and 4 in each stope is assumed to be carried by adjacent rib pillars. Hence the stress on the pillar can be calculated as follows:

\[
\sigma_p = \sigma_h \frac{(L \times T) + (L \times \frac{W_o}{2})}{L \times T}
\]

\[
\sigma_p = \sigma_h \frac{(T + \frac{W_o}{2})}{T}
\]

\[
\sigma_p = \sigma_h \frac{(1 + \frac{W_o}{2T})}{T}
\]

Eq. (3.7)

Where:
- \(\sigma_p\) = Pillar stress, MPa
- \(\sigma_h\) = Horizontal stress, MPa
- T = Crown pillar thickness, m
- L = Length of crown pillar, m
- \(W_o\) = Width of the opening (stope height), m

**Figure 3.4** Modifications of tributary area theory for estimation of the crown pillar stress.
Based on the above assumption the stress in the crown pillar are obtained for different values of stope height to crown pillar thickness ratios from Figure 3.5.

![Figure 3.5](image)

**Figure 3.5** Crown pillar stress/insitu horizontal stress ($\sigma_p/\sigma_h$) versus stope height / pillar thickness ($W_0/T$).

### Chapter 3, Evaluation of Procedure For the Design of a Crown pillar

#### 3.3 Rock mass strength

##### 3.3.1 Size dependent strength

The strength of the rock mass is determined by the strength of the intact rock and by weaknesses in the form of various discontinuities present in the rock. These weaknesses can vary from joints to large faults. The reduction in the strength of the rock is dependent on the number, geometry (persistence, waviness, roughness and orientation in space) and mechanical properties of the discontinuities.

It is important to note that only the discontinuities within the critically loaded area affect the strength of the rock mass (Krauland et al, 1989). Figure 3.6 (Janelid, 1965 sited Krauland et al, 1989) illustrates the conditions in a jointed rock mass. Around a single borehole the critically loaded volume is small, and there are either no weaknesses or only a few weaknesses and the strength is high. The stress redistribution around a tunnel influences a much larger rock volume, containing a large number of structural weaknesses. This results in considerably lower rock mass strength.

At larger sizes of pillars, new types of structural weakness become active such as faults and shear zones as shown in Figure 3.6. These discontinuities result in further weakening of the rock mass.
3.3.2 Methods for strength determination

A variety of methods have been developed in the past for the estimation of the strength of a rock mass. These methods can be divided into four groups:

- Numerical methods
- Rock mass classification systems
- Large-scale testing
- Back analysis methods

Numerical methods: In the numerical methods both the rock substances and the properties of the discontinuities are modelled. Modelling can be done either by simulation of discontinuities as discrete elements of the rock mass or regarding the rock mass as a composite material. All numerical methods require the accurate determination of a large number of input parameters and these may be based on a number of assumptions which can affect the validity of the results.

Rock mass classification systems: The rock mass classification systems can be divided into two groups according to the objectives of the classification (Krauland et al, 1989):

- Stability classification
- Strength classification
In stability classification, the ratio between load and bearing capacity is considered. In other words, the anticipated reaction of the underground structure to this ratio is classified and the necessary support measures are estimated. In this classification the stress field and the influence of the shape of the excavation on the state of stress has been considered implicitly. Such systems can be suitable where there is little variations in the geometry of the excavation, such as in tunnelling.

In strength classification the purpose is to classify the strength of the rock mass only. The load is determined and compared explicitly with the rock mass strength in a stability assessment. Strength classification has been found (Krauland et al, 1989) to be more suitable in mining applications because of large variations in the geometry and size of the excavations and the accompanying states of stress.

Large-scale testing: In large-scale testing, the strength of the entire structural part of rock mass (e.g. pillars) is determined. The results of these tests include the interaction between the rock material and the discontinuities of the rock mass.

Back-analysis methods: Back-analysis of the strength of the rock mass is one way to collect experience on the properties of the rock mass from existing engineering structures. It is more valuable when failure has occurred; in this case the maximum bearing capacity of the rock mass can be determined. This method provides the most reliable strength determination.

Among different methods introduced for of Rock Mass Strength, the rock mass classification systems (chapter 2.2) have the followings advantages which make them suitable for general application at an early stage in a project.

- Economic
- Practical
- Systematic
- Easy to apply

3.3 3 Pillar strength

Sufficient work has not been done in hard-rock pillars for estimating the pillar strength. However, many investigations have been done in coal mines and different formulas have been developed. Basically pillar strength depend on three elements:
(i) The size or volume of the pillar,
(ii) The shape or geometry of the pillar, and
(iii) Strength properties of the pillar material.

(a) The size effect

The concept of critical size (Bieniawski, 1984) is very important in the design of pillars. It means that for cubical specimens of coal, the strength decreases with increasing specimen size until it becomes constant from a critical specimen size onward (Figure 3.7). For South African coal, Bieniawski (1968) concluded that 1.5 m cubic specimens constitute the critical size value. The size effect characterises the difference between the strength of small size specimen tested in the laboratory and the large size pillars which have been created in situ.

![Figure 3.7 Specimen size effect in coal (after Bieniawski, 1984)](image_url)

The following Equations are used to calculate the design parameters in coal pillars (Bieniawski, 1984):

\[
\sigma_1 = \frac{k}{\sqrt{36}} \quad \text{Eq. (3.8)}
\]

Equation 3.8 is for cubical pillars having a height more than 0.9 m.

\[
\sigma_1 = \frac{k}{\sqrt{h}} \quad \text{Eq. (3.9)}
\]
Equation 3.9 is for cubical pillars having a height less than 0.9 m. Holland and Gaddy (1964) showed that the constant 'k' must be determined for the actual pillar material as follows:

\[
\sigma_1 = \frac{k}{\sqrt{D}} \quad \text{Eq. (3.10)}
\]

Where:
\[
\sigma_1 = \text{Uniaxial compressive strength of rock specimens, MPa}
\]
\[
D = \text{Diameter or cube side dimension which is normally between 50 to 100 mm}
\]

(b) The shape effect

Considerable effort has committed throughout the world to develop a method of determining the strength of a mine pillar. Any formula developed should take account of the shape effect as well as the size effect. Some commonly used formulae are given in Table 3.1 (Bieniawski, 1984).

<table>
<thead>
<tr>
<th>Table 3.1 Most commonly used pillar strength formulae</th>
</tr>
</thead>
<tbody>
<tr>
<td>Holland and Gaddy formula, 1964 (\sigma_p = \frac{k \sqrt{w}}{h})</td>
</tr>
<tr>
<td>Salamon-Munro formula, 1967 (\sigma_p = 7.2 \ (w)^{46}/(h)^{66})</td>
</tr>
<tr>
<td>Obert and Duvall formula, 1967 (\sigma_p = \sigma_1 (0.778 + 0.222 \ (w/h)))</td>
</tr>
<tr>
<td>Wilson formula, 1972 (\sigma_p = \sigma_c + \sigma_3 \tan \beta)</td>
</tr>
<tr>
<td>Hedley and Grant formula (for hard rock), 1972 (\sigma_p = k \ (w)^{20}/(h)^{75})</td>
</tr>
<tr>
<td>Holland formula, 1973 (\sigma_p = \sigma_1 \sqrt{\frac{w}{h}})</td>
</tr>
<tr>
<td>Bieniawski formula, 1976 (\sigma_p = \sigma_1 (0.64 + 0.36 \ w/h))</td>
</tr>
</tbody>
</table>

Where:
\[
\sigma_p = \text{Pillar strength}
\]
\[
W = \text{Pillar width}
\]
\[
h = \text{Pillar height}
\]
\[
\sigma_1 = \text{Strength of a cubical specimen of critical size or greater}
\]
\[
\sigma_c = \text{Uniaxial compressive strength}
\]
\[
b = \text{Triaxial stress factor (4 for coal)}
\]
\[
k = \text{Strength of 0.305 m}^3 \text{cubic specimen}
In general most of empirical equations which relate pillar strength to its width and height have been derived from the following formula (Hedley et al, 1980)

\[ \sigma_p = k \frac{W^a}{H^b} \quad \text{Eq. (3.11)} \]

Where:
- \( \sigma_p \) = Pillar strength, psi
- \( W \) = Pillar width, ft
- \( H \) = Pillar height, ft
- \( k \) = Strength of a foot cube
- \( 'a' \) and \( 'b' \) = Constant

Values of constants derived from literature are \( a = 0.50 \) and \( b = 0.50 \) to 1.0. Most work was conducted in coal mine and in one case for hard rock example.

Because the value of \( 'a' \) is relatively constant at 0.5 but the values of \( 'b' \) varies in a range, for converting Equation 3.11 into metric system a coefficient \( \alpha \) was inserted in it (Hedley et al, 1980).

\[ \sigma_p = \alpha k \frac{W^a}{H^b} \quad \text{Eq (3.12)} \]

In metric units \( \alpha \) for value of 'b' equal to 0.75 (hard rock) was calculated as follows (Hedley et al, 1980):

\[ b = 0.75 \quad \alpha = 0.305^{0.25} = 0.743 \]

Therefore, the equation which can be used for hard rock can be rewritten in metric as follows:

For \( b = 0.75 \) \[ \sigma_p = 0.743 k \frac{W^{0.5}}{H^{0.75}} \]

Where:
- \( k \) = Strength of a 0.305 m\(^3\) cubic sample
- \( W \) and \( H \) = Pillar width and height in metres

The mean strength of a 0.305 m\(^3\) cubic sample with uniaxial compressive strength \( (\sigma_c) \) obtained from testing on a rock sample with 54 mm diameter and length to diameter ratio equal 2 is estimated from following equation:
By substitution of Equation 3.13 into the Equation 3.12, the strength of pillars is estimated from the following equation:

\[
\sigma_p = 0.635 \sigma_c \left( \frac{w}{h} \right)^{0.25}
\]

Eq. (3.14)

Using equation 3.14 the pillar strength is presented graphically in figure 3.8 for different values of \( \sigma_c \).

Figure 3.8 Pillar strength versus thickness to span ratios for different values of \( \sigma_c \).

As shown in Figure 3.8, when the thickness to span ratio is less than 2 the strength of pillars are less than 50% of the uniaxial compressive strength of the intact rock. Results of back-analysis of pillars and crown pillars of hard rock in different mines (Sjoberg 1992 and 1990) show that for stable pillars usually the strength of the pillars are more than 50% of the uniaxial compressive strength of intact rock. Stability analysis of the crown pillar (Section 5.6. and 6.6) also indicates that the strength of the stable crown pillars are more than 50% of the uniaxial compressive strength of the intact rock. Also optimisation of crown pillar thickness is one of the objectives of the work undertaken in this thesis and more accurate estimation of the pillar strength especially for low thickness to span ratios is desirable for design purposes, therefore, it seems that the Equation 3.14 underestimates the strength of the crown pillar in the area of interest (low thickness to span ratios) and it is better to examine another criterion for estimation of the pillar strength.
Strength of crown pillars can be estimated using the graph in Figure 3.9. This graph has been created based on Equation 2.13 (Section 2.2.9) for different types of rock masses.

\[
\text{RMS} = \frac{(\text{RMR} - \sigma_c \text{ rating})}{80} \times 0.8 \quad \text{Eq. (2.13)}
\]

![Graph showing Rock Mass Strength versus \( \sigma_c \) for different RMR ratings.](image)

**Figure 3.9** Rock Mass Strength versus \( \sigma_c \) for different RMR ratings.

Equation 2.13 allows the RMR (Rock Mass Rating) to be considered for estimation of Rock Mass Strength. RMR has been reported to be an important factor for the stability of crown pillars (Betournay, 1984).

### 3.4 Investigation of the mode of failure of a cracked roof beam

The voussoir arch theory suggested by Evans (1941) recognises the fact that in a confined situation (beam with fixed ends) the ultimate strength of a beam is greater than its elastic strength. Observations have shown (Wright 1972, Merill 1954) that after excavation of an opening in well-bedded rock, separation will occur at the contacts of bedding planes and tensile cracks will appear at the surface of the excavation as shown in Figure 3.10 (a). Shear failure, which has been predicted for short spans and low confinement stress, is shown in Figure 3.10 (b).
Figure 3.10 Typical modes of roof beam failure; (a) Buckling failure, (b) shear failure

In recent research work on roof bed mechanics Sterling (1980) and Beer and Meek (1982) have formulated the following principal concerning roof rock over mined spans:

- Roof beds can no longer be simulated by continuous elastic beams or plates, since their behaviour is dominated by the blocks or voussoir generated by natural cross joints or induced transverse fractures.

- Roof bed behaviour is determined by the lateral thrusts generated by deflection under gravity loading of the voussoir beam against the confinement of the abutting rock.

- For beams with low span to thickness ratios, the most likely failure mode is shear failure at the abutments.

- Roof span stability is limited by the possibility of buckling for a roof with high span to thickness ratio.

- A roof with low rock material strength, or moderate span to thickness ratio may fail by crushing or spalling of centre or abutment voussoirs.

This techniques assumes that:

- The rock mass behaves as a no-tension medium.
- The rock deforms elastically under compressive stress.
- Shear strength is generated by the frictional resistance due to the horizontal compressive forces acting across the failure planes of the jointed roof rock.
The geometry and mechanics of voussoir arch theory for a typical rock beam are illustrated in Figure 3.11. The roof beam of span, 's', and thickness, 't', supports its own weight, 'W', and each half span of the beam tends to rotate about the abutment section. The compressive stress will be a maximum in the under surface at the abutment, and decreases linearly to zero at some distance up, the section forming a triangular load distribution operating over a depth, nt, of the beam section. A similar but reversed distribution of stress, relative to the beam section, will occur at the midspan. The line of action of the resultant of each distributed loads acts through the centre of the area of this triangle of stress. The initial moment arm of the couple forming the maximum moment of resistance is given by:

\[ z_0 = t \left( 1 - \frac{2n}{3} \right) \]  \hspace{1cm} \text{Eq. (3.15)}

where;

- \( t \) = Thickness of the beam, m
- \( n \) = Ratio of roof beam section under horizontal compression
- \( s \) = Span of roof beam, m
- \( f_c \) = Maximum lateral compressive stress, MPa
- \( W \) = Roof beam weight, MN
- \( z_0 \) = Internal moment of arm of the couple of forces, m

Equilibrium of the roof beam requires that the moment of resistance induced by the couple of force, 'f_c', at the midspan and the abutment balances the moment of forces associated with the weight of the rock, 'W', and the reacting force, 'R', at the abutments. This requires that the following condition be satisfied:

\[ \frac{W}{s^2} = \gamma \cdot s \cdot t \] \hspace{1cm} \text{Eq. (3.16)}

\[ \frac{(\gamma \cdot s \cdot t)}{(f_c / 2) \cdot nt \cdot z_0} \]
\[ f_c = \frac{(\gamma \cdot s^2)}{(4 \cdot n \cdot z_0)} \quad \text{Eq. (3.17)} \]

Where:
- \( s \) = Span of roof beam, m
- \( \gamma \) = Weight density of rock mass, MN/m³
- \( W \) = Roof beam weight with unit length, MN
- \( f_c \) = Maximum lateral compressive stress, Mpa

When the beam deflects, the compression arch is shortened, and the lateral thrust finally mobilised so that which allows Equation 3.17 to be satisfied. The relation between mobilised thrust and beam deformation exploits the assumption of the shape of the arch (or initial thrust line) operating longitudinally in the beam. It is assumed the arch profile is parabolic with the following Equation:

\[ s'^2 = 4a z \quad \text{Eq. (3.18)} \]

Where \( s' \) is the beam half span and 'a' is chosen to satisfy the known geometry. Using analytical geometry and the theory of parabolic functions the length of the arch can be calculated as follows:

The length of a smooth arch given as a graph \( y = f(x) \) for \( x \) in \([a, b]\) is obtained by the following formula (Edward and Penney, 1982):

\[
\text{Arch length} = \int_{a}^{b} \sqrt{1 + [f'(x)]^2} \, dx = \int_{a}^{b} \sqrt{1 + \left(\frac{dy}{dx}\right)^2} \, dx \quad \text{Eq. (3.19)}
\]

When \( dy/dx \) is very small ie. the roof span 's', is very large in comparison with voussoir arch height (z), the expression \((1 + (dy/dx)^2)^{0.5}\) can be expanded as follows:

\[
\sqrt{1 + \left(\frac{dy}{dx}\right)^2} = (1 + \left(\frac{dy}{dx}\right)^2)^{\frac{1}{2}} = 1 + \frac{1}{2} \left(\frac{dy}{dx}\right)^2 \quad \text{Eq. (3.20)}
\]

Therefore, the arch length 'L' is obtained from the following relation:

\[
L = \int_{a}^{b} 1 + \frac{1}{2} \left(\frac{dy}{dx}\right)^2 \, dx
\]
Equation 3.21 as shown below is the general formula for a parabola (Edward and Penney, 1982). From this equation the expression \( \frac{dy}{dx} \) is obtained.

\[
y = px^2 \quad \text{Eq. (3.21)}
\]

and; \( \frac{dy}{dx} = 2px \)

Now, arch length 'L' is calculated from the following relation:

\[
L = \int_a^b \left[ 1 + \frac{1}{2} (4p^2x^2) \right] dx
\]

or;

\[
L = \left[ x + \frac{2}{3} p^2 x^3 \right]_a^b
\quad \text{Eq. (3.22)}
\]

When 'x' is equal to half of the roof span, 'y' becomes equal to the height of the arch, 'z_0'. In this case the value of 'p' in Equation 3.21 is obtained.

\[
p = 4z_0/s^2
\]

Substitution of 'p' into equation 3.22 gives the length of the arch in terms of its height, 'z_0', and span 's' by:

\[
L = \left[ x + \frac{2}{3} p^2 x^3 \right]_{-s/2}^{s/2} = 2 \left[ x + \frac{2}{3} p^2 x^3 \right]_{0}^{s/2} = 2 \left[ \frac{s}{2} + \frac{2}{3} p^2 \frac{x^3}{8} \right]
\]

\[
L = s + 1/6 p^2 s^3
\]

\[
L = s + \frac{8z_0^2}{3s}
\quad \text{Eq. (3.23)}
\]

When the beam deflects the compression arch will shorten and if the shortening of the arch is \( \Delta L \), the new lever arm 'z' is:

\[
z = \sqrt{\frac{3s}{8} \left( \frac{8z_0^2}{3s} - \Delta L \right)}
\quad \text{Eq. (3.24)}
\]

where:

\( \Delta L = \) Incremental shortening of the rock arch, m
The value of $\Delta L$ is the combination of the elastic shortening, abutment yield and inelastic shortening. An average value for the elastic shortening is obtained from the following Equation:

$$\Delta L = \frac{11}{24} \frac{f_c}{E} L$$

Eq. (3.25)

Where:

$E = \text{Elastic modulus of the rock beam, GPa}$

Abutment yield and inelastic shortening may be simply considered by reducing the value of $E$. Substitution of $z$ into Equation 3.17 gives a fourth-order equation for $s'$ as a function of $n'$ and $f_c'$ as follows (Beer and Meek, 1982):

$$s'^4 + C_1 s'^2 - C_2 = 0$$

Eq. (3.26)

$$C_1 = 0.178 A^2 f_c' / E$$

Eq. (3.27)

$$C_1 = A^2 z^2_0 (1 - (11 f_c' / 24 E))$$

Eq. (3.28)

$$A = \frac{4 \cdot n \cdot f_c}{W}$$

Eq. (3.29)

Where; $C_1$, $C_2$ and $A$ are variables, $f_c'$ is the maximum lateral compressive stress and $W$ is the roof beam weight.

This equation can be solved for different values of $f_c'$. The value of $s'$ first will increase and reaches a maximum at a particular critical stress ($f_{cr}$) and then decreases (Beer and Meek, 1982). For low values of beam thickness $f_{cr}$ is lower than the compressive strength of the rock and in this case the stability of the voussoir beam is dependent on the modulus of elasticity only. For larger values of $t$ the stability may also depend on the compressive strength of the rock. The horizontal compressive stress, $f_c'$, which acts through the beam section changes as a result of beam deflection and arch shortening, therefore, the depth ratio ($n$) of the compressive stress is:

$$n = (3/2) \cdot (1 - z / t)$$

Eq. (3.30)
For solution of this equation a sequential calculation of the arch parameters for different values of load to depth ratio which starts from \( n = 0 \) to \( n = 1 \) (over the whole depth of the beam section) is needed. When the solution starts it will continue until stable values of the maximum horizontal stress are obtained. Results of calculation have shown that a maximum stable horizontal compressive stress is obtained when \( n = 0.75 \) (Bensehamdi, 1989). The value of the maximum lateral stress, \( f_c' \), increases when 'n' increases to the value of 0.75, at which \( f_c' \) reaches its maximal stable value. The value of \( f_c' \) starts to decrease after this maximum for the values of 'n' more than 0.75.

The stability of the roof strata is assessed by considering each of the possible modes of roof failure. Roof failure in compression begins by lateral rock crushing at points of high compressive stress, in this case the maximum lateral compressive stress, \( f_c' \), should be compared with the uniaxial compressive strength of the rock material forming the roof slab. Shear failure in the roof beam occurs if the shear stress, \( \tau_{\text{max}} \), at any location exceeds the shearing resistance developed by the horizontal compressive stress. The condition for failure can be simply described by the Mohr-Coulomb criteria:

\[
\tau_{\text{max}} = f_c \cdot nt \cdot \tan \phi + C \quad \text{Eq. (3.31)}
\]

Where:
- \( \tau_{\text{max}} \) = Shear strength of the rock, MPa
- \( f_c \) = Lateral compressive stress, MPa
- \( nt \) = Load depth
- \( \phi \) = Friction angle, degrees
- \( C \) = Cohesion, Mpa

The maximum shear stress \( T_{\text{max}} \) is induced at the abutment and is equal to:

\[
T_{\text{max}} = \frac{W}{2 \cdot A} \quad \text{Eq. (3.32)}
\]

where; \( A \) = Cross sectional area, m\(^2\)

The most critical situation in which shear failure can occur is when the major principal stress, \( \sigma_1 \) or the parameters 's' or 'A' (area on which shear stress is active) is small. When there is a short span or low confining condition \( \sigma \) is small because the voussoir arch cannot be formed. Buckling of the roof, which is another mode of roof failure, arises when the beam deflects and the compression arch is shortened, and its height, 'z', becomes negative. A typical mode of roof failure by compression and shear action was shown in Figure 3.11.
3.4.1 Square plates

Voussoir beam theory was extended to plates by applying the concept of yield lines which is a concept which has been used in the design of reinforced concrete. By this the rib pillars and abutments can be considered for stability of the hangingwall. It is assumed that tension cracks have developed at the edge and along diagonals (Figure 3.3 (a)). The expression for moment equilibrium in this case can be written as follows:

\[ \frac{\gamma s^3 t}{24} = \frac{f_c n t}{2} s z \]

Eq. (3.34)

The left hand side of Equation 3.33 is one-third of the value that was obtained for the beam by equation 3.17. Derivation of the other design formulae are similar to the beam. At the time of computation of the change in length of the arch it should be considered that the state of stress is biaxial. For this case elastic shortening is obtained form the following equation:

\[ \Delta L = \frac{11 f_c L}{24 E (1-v)} \]

Eq. (3.34)

Where; \( v = \) Poisson's Ratio

3.4.2 Rectangular plates

For rectangular plates the theory is more complicated. The pattern of yield lines must be computed first (Figure 3.12 (b)). Distance 'x' can be determined from the condition of equilibrium between external and internal forces as follows (Brady and Brown, 1985):

\[ x = \frac{a}{2} (\sqrt{\gamma k^2 + 3} - k) \]

Eq. (3.35)

where; \( k = \frac{a}{b} = \) The ratio of short span to long span
Chapter 3, Evaluation of Procedure For the Design of a Crown pillar

3.4.3 Prestress and stress redistribution in rock around a mine opening

Virgin stress conditions at any point in a rock mass are the result of the load caused by the superincumbent strata, related 'poisson effects', and also by various geological events. By overburden effects alone the horizontal stress $\sigma_h$, would be around 1/3 to 1/4 the vertical stress, $\sigma_v$. In Australia $\sigma_h$ can be up to 4 $\sigma_v$ due to 'locked-in' horizontal stresses. This can lead to virgin horizontal stress magnitudes of up to 60 MPa at 500 m; an average mining depth. After creating an opening the virgin stresses are redistributed to the rock mass surrounding the opening, resulting in stress concentrations at the abutments. The pattern of stress redistribution can be determined by numerical analysis.
and photoelastic modelling techniques. In the case of a crown pillar the horizontal stress field and its redistribution is taken as inducing a pre-stress condition in the pillar.

In order to illustrate the principle of prestress Corlett & Emery (1959) considered the situation of a simple horizontal beam of five sections, which were jointed but not coupled to each other, Figure 3.13. The following conditions are pertinent to Figure (a):

- The beam cannot support its own weight, and
- The joints open under tension and the beam fails.

\[ \begin{align*}
\text{dead load} & \quad \text{pre-stress} \quad \text{total unloaded load} \\
0 & \quad 3600 & \quad 3600 & \quad 7200 \\
\text{c-beam subjected to axial force} & \\
0 & \quad 7200 & \quad 7200 & \quad 7200 \\
\text{d-beam subjected to axial force at a point 1/3d from the lower face} &
\end{align*} \]

**Figure 3.13** Stress diagram in a prestressed beam
With a uniform load of 1.83 KN/m, the moment at the mid span produces a maximum stress of 3600 KPa which would cause failure under the above conditions. If the beam is subjected to an axial stress of 3600 KPa and the dead load is assumed to be zero, the uniform load of 1.83 KN/m can now be supported (Figure 3.13 'b' and 'c'). Now if the point of action of the end load is moved to 1/3d from the lower face, the pre-stress will now withstand a uniform load of 3.66 KN/m as shown in Figure 3.13 (d).

3.4.4 Design curves for cracked roof beams

Beer and Meek (1982) used the voussoir beam theory to present a series of design curves for roof and hanging-wall spans of excavations in bedded rock. These curves assume plain strain conditions and give conservative estimates of the stable span for an underground opening (Figure 3.14).

![Figure 3.14](image)

**Figure 3.14** Maximum span for a self supporting roof beam as a function of strata thickness (after Beer and Meek, 1982)

Based on the voussoir beam theory a simple computer program has been written (appendix 1) to solve the sequential Equations for rock beams, square and rectangular plates and check for safety against shear, compression or buckling failure. The program allows for the introduction of confining stress based on prestressed rock beams (Section 3.4.3). A typical procedure for solution is shown in Figure 3.15.
Figure 3.15 Roof design procedure based on voussoir beam and plate principles

where;

- $t =$ Thickness of the beam, m
- $n =$ Ratio of roof beam section under horizontal compression
- $f_c =$ Maximum lateral compressive stress, MPa
- $z_0 =$ Internal moment of arm of the couple of forces, m
- $s =$ Span of roof beam, m
- $\gamma =$ Weight density of rock mass, MN/m$^3$
- $W =$ Roof beam weight with unit length, MN
- $\Delta L =$ Incremental shortening of the rock arch, m
- $E =$ Elastic modulus of the rock beam, GPa
- $\tau_{\text{max}} =$ Shear strength of the rock, MPa
- $n_t =$ Load depth
- $\phi =$ Friction angle, degrees
Results from the program indicated that when confining stress is negligible (a crown pillar near the surface) the stable zone for safe spans of excavations is situated between areas of shear and buckling failure as shown in Figure 3.16. It may be noted that with an increase in thickness the possibility of shear failure will increase, whereas buckling failure occurs when the thickness to span ratio is low. A most interesting point is that when a jointed beam is short it may be unstable but by increasing the span it will enter a zone of stability. This is because the voussoir arch has not been formed at that short span. This fact is illustrated clearly in Figure 3.16 and is consistent with previous work on arching action in cracked roof beams (Wright, 1972).

![Figure 3.16](image_url)

**Figure 3.16** Safe span for a self supporting roof beam as a function of strata thickness no confining stress.

Figure 3.16 shows the factor of safety against shear failure as a function of the thickness to span ratio for different joint friction angles. It is clear that when the thickness to span ratio is high the mode of failure is shear failure. For low thickness/span ratios, the potential mode of failure is either compression or buckling.
**Figure 3.17** Factor of Safety against Shear failure as a function of the thickness/span ratio for different joint friction angles.

By increasing the lateral confining stress the factor of safety against shear, as shown in Figure 3.18, improves considerably. It is clear that with the existence of confining stress the potential mode of failure has changed to compression failure and the stability of the span is dependent on the crushing strength of the rock, $\sigma_c$.

**Figure 3.18** Factor of safety against shear failure as a function of the thickness/span under different confining stresses.

The minimum thickness for the crown pillar is chosen by using the nomographs presented in Figures 3.19 to 3.24. These graphs are based on voussoir beam and plate theory and
have been modified (Section 3.2) to show the minimum thickness of beam or plate before buckling of a rock with specific elastic modulus (E), under a given stress regime or no-confining stress.

**Figure 3.19** Safe span for self supporting square roof as a function of strata thickness - no confining stress

**Figure 3.20** Safe span for a self supporting roof beam as a function of strata thickness based on voussoir beam theory.
Figure 3.21 Safe span for self supporting square roof as a function of strata thickness based on voussoir plate theory.

Figure 3.22 Safe span for a self supporting rectangular roof as a function of strata thickness (S/L = 0.5).
3.5 Procedure for crown pillar design method

(1) The safe span of a crown pillar can be obtained using different rock mass classification systems or stability graph method (Mathews et al, 1981). Stability graph method was developed for stope design and has been used successfully for this purpose (Bawden et al, 1989).
(2) The minimum thickness of the crown pillar is chosen by using the nomographs presented in Figures 3.19 to 3.24.

(3) The stress in the crown pillar is obtained for different values of stope height to crown pillar thickness ratio from Figure 3.5.

(4) Strength of the crown pillar is obtained from Equation 2.13 or the graph in Figure 3.9.

(5) The factor of safety is obtained from strength/stress ratio in the pillar.

3.6 Discussion

A structure is generally defined to be stable when the factor of safety is greater than one. In engineering practice the magnitude of the factor of safety is dependent on the knowledge of the condition of the structure, i.e., the greater the knowledge or the condition of the structure, the lower the acceptable factor of safety. It is important to know that when the factor of safety is one the probability of failure is 50% (Singh and Eski, 1987). The factor of safety should be greater than one to achieve a low probability of failure. If the factor of safety is more than one the crown pillar is stable. Otherwise two alternative are suggested:

(a) Increasing the thickness and estimating the stress in the crown pillar by using Figure 3.5. Then comparing the new stress with the RMS until an acceptable value for the factor of safety is obtained.

(b) Improving the pillar strength by supporting the crown pillar using different bolt factors as shown in Figures 3.9 and 3.25. The graph in Figure 3.25 is based on the original graph developed by Potvin et al. (1989) and replotted here in RMR units for convenience. It can be seen from Figure 3.25 that application of cable bolts improves RMR rating in the crown pillar. Because Rock Mass Strength (RMS) is a function of the Rock Mass Rating (RMR), therefore, with an increase in RMR the strength of the pillar will also increase. By comparing the strength and stress of a supported crown pillar a new factor of safety is obtained. If the factor of safety is more than one the crown pillar will be stable otherwise the above procedure should be repeated until an acceptable value for the factor of safety is obtained. The length of the cable bolts for support of crown pillars are obtained from Figure 2.20.
3.7 Conclusions

The objective of this chapter was to review various methods of optimisation of crown pillar design based on a factor of safety approach. The stability of a crown pillar depends on, amongst many factors, the stope geometry, rock properties and the magnitude of the lateral stress. An increase in confining stress allows for a greater span, but only to a limit. If confining stress is excessive the pillar will fail in compression; which is often dynamic in nature. Confining stress also reduces the risk of shear failure at lower thickness to span ratios. This is significant in surface crown pillar design where lateral stresses are small.

In general a method for the design of a crown pillar has been proposed in which stress and strength of crown pillars are estimated from suggested design charts. Then, the factor of safety is obtained from the strength/stress ratio in. If the factor of safety is more than one the crown pillar is stable and if it is less than one, either the thickness is increased (in order to reduce the stress in the pillar) or strength is improved by bolting the crown pillar.
A combination of the two options can also be used if necessary. The procedure for crown pillar design using the above method is shown in Figure 3.26.

Figure 3.26 Procedure for crown pillar design
CHAPTER 4

RESEARCH TECHNIQUES

4.1 Introduction

This chapter presents two basic techniques for data acquisition and data reduction for the design of crown pillars in two Australian metal mines. In the first part of this chapter field techniques for studying the regional and mine geology as well as joint surveying have been discussed. Structural data have presented and reduced to enable the determination of potential for rock wedge failure in underground excavations. This helps to obtain an initial engineering understanding of the rock mass structure.

The second part of this chapter presents rock testing techniques for obtaining strength and elastic properties of rock samples for the design of structures in rock. Laboratory testing of rock is one of the most important parts of engineering work in the field of rock mechanics. Some rock tests can be performed both in the laboratory or in the field. This includes the determination of uniaxial and direct shear strength, but triaxial tests must normally be performed in the laboratory.

4.2 Field techniques

Field techniques deal with the evaluation of various important properties of discontinuities influencing the engineering behaviour of rock masses. These include identification of the types of structural features occurring in the rock mass and methods of collecting, analysing and presenting structural data.
4.2.1 Structural defects

The structural defects of a rock mass are the result of orogenic movements in the earth's crust and they play an important role in the stability of the excavations, selection of the mining method employed and design of support. The structural defects of rock masses and their engineering properties are discussed below (Brady and Brown, 1985).

**Bedding** planes separate sedimentary rocks into beds or strata and are generally very persistent. Bedding planes can contain material of different grain size from the sediments which form the rock mass or they may have been partially influenced by low-order metamorphism. In each of these cases there will be cohesion between the beds.

**Folds** are continuous curved surfaces formed from the deformation of pre-existing planar surfaces. Natural folds are very large in shape and size. Folds are classified according to their geometry and method of formation. The main effect of folds is that they alter the orientation of beddings locally and are a determining factor in the creating other standard features. For example joint sets may be formed in the crest or through a fold. Figure 4.1 shows a typical development of jointing in a stratum on an anticline. When the sedimentary rocks are folded, shear stresses develop between the beds where slip may occur. Fracture cleavage may also develop as a series of closely spaced fractures as a result of shear stresses which are associated with folding.

![Figure 4.1](image)

**Figure 4.1** Typical development of joints in a folded stratum

**Faults** are fractures and rock can move across this fracture in a direction which is generally parallel to the fault surface. Faults may intersect entire mining areas or may occur in a small
area. Faults thickness varies from millimetres to tens of metres and may contain weak infilling materials such as fault gouge (clay), fault breccia (recremented) and angular fragments. The contact rock may be slickensided and may be coated with minerals such as graphite and chlorite which have low frictional resistance. Faults are zones of low shear strength through which slip may easily occur.

Shear Zones are areas in which a ductile deformation or distortion has occurred. Fracture surfaces in a shear zone may be slickensided or coated with low-friction materials. Shear zones like faults have low shear strength, but are more difficult to identify visually.

Joints are fracture surfaces along which no movement has occurred. Normally joint orientations are systematic. Joints may be open, filled or healed. They frequently form parallel to bedding planes, foliation or cleavages.

4.2.2 Geomechanical properties of discontinuities

The term 'discontinuities' is used by engineers and geologists as a common term for fractures and features in a rock mass such as joints, faults, shear zones, weak bedding planes and contacts that have very low tensile strength. Various important properties of discontinuities that influence the engineering behaviour of rock mass are discussed here. For a full description of these properties "Suggested Methods of Quantitative Description of Discontinuities" prepared by the Commission on Standardisation of Laboratory and Field Tests, International Society of Rock Mechanics [1978 and 1981] can be consulted.

Joints or discontinuities affect the mechanical behaviour of the rock mass considerably firstly by their nature and secondarily, they reduce the rocks resistance to weathering. So it is necessary to consider the structure of the rock mass as well as the discontinuities carefully. The following characteristics of discontinuities are most important ones which influence the rock mass behaviour.

(a) Orientation

The orientation or attitude of a joint or discontinuity in space is described by the dip of the line of steepest declination from horizontal and by the dip direction measured clockwise from true north. Orientation can be measured by compass and clinometer or photogrammetry methods. It is desirable to measure a sufficient amount of orientation to define the various joint sets of given domains. By representation of the orientation information in block diagrams or spherical projection and analysis of them, the effect of different sets on each other and the probable type of failure plane can be provided. Figure 4.2 shows the strike, dip and dip direction of a joint plane.
Figure 4.2 Diagram showing the strike, dip and dip direction of a joint plane

In Rock Mechanics, it is usual to record orientation data in the form of dip direction (3 digits) / dip (2 digits). The orientation of discontinuities relative to the faces of an excavation plays an important role in instability problems that may arise as a result of blocks of rocks along discontinuities. The orientation of the various joint sets will determine the size and the shape of these blocks.

(b) Spacing

The spacing is the perpendicular distance between adjacent discontinuities and is usually expressed as the mean spacing of a particular set of joints. It often determines the size of blocks that make up the rock mass. Priest and Hudson (1981) presented a probability density distribution that can be approximated by the negative exponential distribution. Therefore, the frequency, \( f(x) \), of a given discontinuity spacing value, \( x \) is given by the function:

\[
f(x) = \lambda \cdot e^{-\lambda x}
\]

Eq. (4.1)

where; \( \lambda = 1 / \bar{x} \) is the mean discontinuity frequency and \( \bar{x} \) is the mean spacing.

Their findings have been verified for a wide range of rocks and allows their probability function to be used for predicting block size and possible intersections. Deere (1964) proposed a system of classifying a rock mass using RQD (Rock Quality Designation) for quantifying discontinuity spacing. RQD is determined from drill core and is given by the following equation:
\[ RQD = \frac{100 \sum x_i}{L} \]  
Eq. (4.2)

where; \( x_i \) is the length of individual pieces of core which have a length of 0.1 m or greater and \( L \) is the total length of the core.

Priest and Hudson (1981) found that an estimate of RQD could be obtained from discontinuity spacing measurements made on core or an outcrop by using the following empirical relationship:

\[ RQD = 100 e^{-0.1\lambda} (0.1 \lambda + 1) \]  
Eq. (4.3)

For values of \( \lambda \) in the range of 6 to 16 per metre, a good approximation to measured RQD values was found to be given by the following linear relationship (Brady and Brown, 1985):

\[ RQD = -3.68 \lambda + 110.4 \]  
Eq. (4.4)

The following terminology can be used for describing the spacing (Table 4.1):

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

(c) **Persistence**

Persistence is the aerial extent or the size of the discontinuities within a plane, Figure 4.3. In general terms it can be quantified by observing the trace length of discontinuities on exposed surfaces. Although persistence is one of the most important rock mass parameters, it is the most difficult to determine. Persistence of discontinuities has a major influence on the shear strength developed in the plane of a discontinuity and also on the fragmentation
characteristics, cavability and permeability of the rock mass. Table 4.2 is the classification of discontinuities according to the most common trace lengths (ISRM, 1981).

**Figure 4.3** Persistent and non-persistent discontinuity (after ISRM, 1987).

**Table 4.2** Classification of discontinuities according to their persistence (after ISRM, 1981)

<table>
<thead>
<tr>
<th>Description</th>
<th>Modal trace length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very low persistence</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>Low persistence</td>
<td>1 - 3</td>
</tr>
<tr>
<td>Medium persistence</td>
<td>3 - 10</td>
</tr>
<tr>
<td>High persistence</td>
<td>10-20</td>
</tr>
<tr>
<td>Very high persistence</td>
<td>&gt;20</td>
</tr>
</tbody>
</table>

(d) **Joint Roughness**

Roughness is a measure of inherent surface roughness and waviness relative to the mean plane of the discontinuity. Both roughness and waviness contribute to shear strength. Large scale waviness may also alter the dip locally. Obviously the importance of wall roughness declines as aperture, or filling thickness, or the degree of any previous displacement increases. The purpose of roughness sampling is for the estimation of shear strength and dilation. Table 4.3 and Figure 4.4 show the classification of discontinuities according to their roughness.
### Table 4.3 Classification of discontinuity roughness (after ISRM, 1981)

<table>
<thead>
<tr>
<th>Class</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Rough or irregular, stepped</td>
</tr>
<tr>
<td>II</td>
<td>Smooth, stepped</td>
</tr>
<tr>
<td>III</td>
<td>Slickensided, Stepped</td>
</tr>
<tr>
<td>IV</td>
<td>Rough or irregular, undulating</td>
</tr>
<tr>
<td>V</td>
<td>Smooth, Undulating</td>
</tr>
<tr>
<td>VI</td>
<td>Slickensided, Undulating</td>
</tr>
<tr>
<td>VII</td>
<td>Rough or irregular, planar</td>
</tr>
<tr>
<td>VIII</td>
<td>Smooth, planar</td>
</tr>
<tr>
<td>IX</td>
<td>Slickensided, planar</td>
</tr>
</tbody>
</table>

**Figure 4.4** Typical roughness profiles (after ISRM, 1981).
(e) Aperture

Aperture is the perpendicular distance separating the adjacent rock walls of an open discontinuity in which the intervening space is filled with air or water. Aperture size plays an important role in shear strength. Aperture can affect the shear strength as well as the permeability or hydraulic conductivity of a discontinuity. The classification and definition of aperture are shown in Figure 4.5 and Table 4.4.

Figure 4.5 Definition of the aperture of open discontinuities and the width of filled discontinuities.
Table 4.4 Classification of discontinuities according to their aperture
(after ISRM, 1981)

<table>
<thead>
<tr>
<th>Aperture (mm)</th>
<th>Description</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 0.1</td>
<td>Very tight</td>
<td>&quot;Closed&quot; features</td>
</tr>
<tr>
<td>0.1 - 0.250</td>
<td>Tight</td>
<td></td>
</tr>
<tr>
<td>.25 - 0.5</td>
<td>Partly open</td>
<td></td>
</tr>
<tr>
<td>0.5 - 2.5</td>
<td>open moderately</td>
<td>&quot;Gapped&quot; features</td>
</tr>
<tr>
<td>2.5 - 10</td>
<td>Wide</td>
<td></td>
</tr>
<tr>
<td>&gt;10</td>
<td>Wide</td>
<td></td>
</tr>
<tr>
<td>10 - 100</td>
<td>Very wide extremely</td>
<td>&quot;Open&quot; features</td>
</tr>
<tr>
<td>100 - 1000</td>
<td>Wide</td>
<td></td>
</tr>
<tr>
<td>&gt;1000</td>
<td>Cavernous</td>
<td></td>
</tr>
</tbody>
</table>

(f) Filling

Filling is the material separating the adjacent rock walls of a discontinuity. Common filling minerals are; calcite, chlorite, clay, silt, fault gouge, breccia, quartz or pyrite. Although some discontinuities are filled with strong vein materials (calcite, quartz, pyrite), filled discontinuities generally have lower shear strength than comparable clean and closed ones. The behaviour of a filled discontinuity depends on a wide range of properties of filling materials such as:

- Mineralogy
- Particle size
- Water content and permeability
- Shear displacement
- Wall roughness
- Width of filling
- Fracturing, crushing or chemical alteration of wall rock

(g) Block size

Block size is a function of discontinuity spacing and the number of joint sets. Individual discontinuities may further influence the block size and shape. The rock deformability will increase as the block size of the rock mass gets smaller. Rock masses can be described based on their block size as shown in Table 4.5.
Table 4.5 Description of rock masses based on block size (after ISRM, 1981)

<table>
<thead>
<tr>
<th>Name</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive</td>
<td>Few joints or very wide spacing</td>
</tr>
<tr>
<td>Blocky</td>
<td>Approximately equidimensional</td>
</tr>
<tr>
<td>Tabular</td>
<td>One dimension considerably smaller than the other two</td>
</tr>
<tr>
<td>Columnar</td>
<td>One dimension considerably larger than the other two</td>
</tr>
<tr>
<td>Irregular</td>
<td>Wide variation of block size and shape</td>
</tr>
<tr>
<td>Crushed</td>
<td>Heavily jointed to &quot;sugar cube&quot;</td>
</tr>
</tbody>
</table>

4.2.3 Joint surveying

Joint surveying is a systematic method of collecting and evaluating joint data. From this data a reliable model which is representative of the joints of the rock mass is constructed and from which the nature of the joint and their distribution can be analysed. There are two method of joint surveying:

1. Line sampling method: In this method a line or measuring tape is stretched along an exposed surface and every joint that intersect this line or tape is measured.

2. The area sampling method: This is another form of joint survey in which all the joints in a selected area of exposure should be measured. In practice this method is more difficult because it is two dimensional, however, where the line sampling method is not desirable, the area sampling method can be used.

In the present study the line sampling method has been used. The basic technique used in mapping underground exposures using scanline survey has been described by Brady and Brown (1985). A scanline is a line set along the surface of the rock mass (Figure 4.6), and from which for all discontinuities intersecting the scanline the following data is recorded:

(a) Distance along scan line, from some datum, to the point of intersection with each discontinuity (D in Figure 4.6).
(b) Dip direction/dip of each discontinuity, measured with a magnetic compass or geological compass.
(c) Is discontinuity open and by how much.
(d) If infilling material is present in open discontinuities.
(e) If water is present.
(f) Estimate of the nature of the discontinuity of the rock exposure surfaces.
Waviness and surface roughness of the same surfaces.

(g) The length of the discontinuity along the scanline (L in Figure 4.6).

(h) Nature of the termination point (A = at another discontinuity, I = in rock material, O = obscured or extending beyond the extremity of the exposure)

Figure 4.6 Scanline survey

After establishment of the scanline, the location, date, rock type, face orientation and scanline orientation should also be recorded in the logging sheet.

Surveying equipment

The following equipment is required for doing a scanline survey:

1. Geological compass
2. Measuring tape
3. Nails
4. Twine
5. Steel measuring tape
6. Camera
7. Survey log book
8. Hammer
The geological compass is used by placing the folding lid against the plane to be measured and the body of the compass should be levelled with the target bubble. The dip and dip direction of the plane may then be read. The tape is used to measure the line along which the survey has been carried out. A survey peg is a suitable reference point.

![Diagram of geological compass](image)

**Figure 4.7** General view of geological compass used in joint surveying

**Source of error in joint surveys**

Errors in joint surveying can originate from either measurement or when trying to identify the various discontinuities. Errors occurring with measurement appear to be associated with the measurement of dip direction such as:

(i) Errors from the presence of metal near a magnetic compass.
(ii) Error in reading dip direction of planes with a high angle of dip.
(iii) Poor levelling of the compass;

Errors in identification arise from:

(i) Disregarding small discontinuities.
(ii) Large and continuous fracture surfaces can be measured several times.
(iii) Discontinuities which are parallel to the excavation surface may be ignored.

**4.2.4 Presentation of structural data**

There are several methods for presenting and analysing of data collected from joint surveying. One method which has been widely used for this purpose is stereographic
projection. Using the stereographic method, ground conditions and also the size and shape of potentially unstable rock wedges and their mode of failure can be identified in an underground excavation. However, the technique is not suitable for more than three discontinuity sets (Hoek and Brown, 1980).

When a large number of discontinuity data are recorded the direction of the pole of each discontinuity will be recorded on a stereographic net. There are two kinds of stereographic net, lower hemisphere and upper hemisphere projection, however, the lower hemisphere is usually used in rock mechanics (Hoek and Brown, 1980). Two principal types of projection are used to present discontinuity data on a reference plane, these are illustrated in Figure 4.8.

![Figure 4.8](image.png)

**Figure 4.8** (a). Equal area projection. (b). Equal angle projection.

In this thesis for presentation of structural data the lower hemisphere equal area projection has been used. In this kind of projection a unit area anywhere in the projection is representative of the same function of the total area of the reference hemisphere. Figure 4.9 illustrates the polar stereographic net.

(a) Method of contouring pole plots

In order to count and contour the poles, several methods have been suggested. Some of these methods are manual and some of them are available to computer processing. For manual counting of poles the Denness curvilinear cell counting method has been used (Denness, 1972). This method has some advantages over other methods particularly when used with pole concentrations very close to the circumference of the net. Denness devised a counting method in which the reference sphere is divided into 100 squares.
A 1\% counting square on the surface of the reference sphere, marked A in Figure 4.10, projects onto the equal area stereonet as a curvilinear cell, Figure A'. When the counting cell falls across the equatorial sphere only the poles falling in the lower half of the 1\% cell will be shown on the stereonet because only the lower part of the reference sphere is used in the plotting. The counting cell marked B and its projection B' show this situation. Poles which fall above the equator are plotted on the opposite side of the stereonet and therefore, the count of the total number of poles which falls within the 1\% square which is situated across the equator is obtained by summing the poles in the shaded portions of both projections marked B'.
Computer programs are also available for processing structural geology data and are used by many civil and mining engineering specialists. Both manual and computer methods have been used in the present study.

(b) Recommended contouring procedure

The following procedure has been recommended (Hock and Brown, 1980) for accuracy in contouring pole plots:

- Use a Denness type B counting net (Figure 4.11) to count the number of poles which falls in each counting cell.

- Sum these individual counts to obtain the total number of poles plotted on the net and write down the number of poles per 1% area which correspond to the different contour percentage values.

- Draw very rough contours on the basis of the pole counts noted on the tracing paper.

- Use the circle counter to draw the contour, starting with low value contour towards the maximum pole concentrations.

4.2.5 Determination of the possible mode of failure of rock wedges formed in underground excavations

Lucas (1980) presented a stereographic method for the determination of the possible mode of failure of a tetrahedral rock wedge, either in a rock slope or bounding an underground excavation. The method considers non-rotational modes of failure and a table is given to facilitate failure mode identification. The following method allows for the presence of three bounding planes and an excavated face and the failure mode is predicted for any tetrahedral wedge having any orientation.

For any underground wedge, 'A', 'B', 'C' are appointed as the three enclosed planes and F is the excavated face as shown in Figure 4.12. The vector normal to the planes i.e. the poles to the plane, are considered to be 'A', 'B' and 'C' and are positive in an upward direction. \( \mathbf{I}_{ab} \), \( \mathbf{I}_{ac} \) and \( \mathbf{I}_{bc} \) represent the intersection vectors (lines of intersection) of planes 'A' and 'B', 'A' and 'C', and 'B' and 'C' respectively. The intersection vector is considered to be positive in a direction from the wedge apex towards the excavated face. Vector azimuths are measured from 0-360° and their inclinations from -90° (vertically...
upwards) to +90° (vertically downwards). The intersection vector which has the most positive dip is now designated $I_{xy}$, which is the line of intersection of planes $X$ and $Y$. The third plane is appointed to be plane $Z$.

There are three possible modes of failure for wedges:

Recumbent wedge: If all the intersection vectors have negative dips then the wedge is recumbent.

Falling wedge: If the azimuths of the three intersection vectors don't lie within 180° of arc, the wedge can then fall vertically.

Sliding wedge: In order to determine uniquely the mode of failure of a sliding wedge, three comparisons of the azimuths of the intersection vectors and the plane vector normals are carried out in order to determine uniquely the mode of failure. In these comparisons the following azimuths are involved:

$$I_{xy}, X \text{ and } Y$$ (i)
$$I_{xy}, X \text{ and } I_{xz}$$ (ii)
$$I_{xy}, Y \text{ and } I_{yz}$$ (iii)

In each group, the vector whose azimuth is between the other two is noted. From these comparisons there are three possible outcomes for case (i) and two each for case (ii) and (iii), and twelve possible outcomes for the three comparisons considered together. From a consideration of these outcomes the mode of failure of the wedge can be determined from the following procedure.

Figure 4.12 Definition of the planes composing an underground wedge.
(1) Plot the pole to planes 'A', 'B', 'C' and the excavated face 'F' on the stereogram as upper hemisphere projection.

(2) Draw the great circle described by the excavated face in upper hemisphere projection.

(3) Construct the intersection vectors $I_{ab}$, $I_{bc}$ and $I_{ac}$ by drawing the pole to the great circle which passes through the poles to the pair of planes defining each intersection. All the intersection vectors are regarded as positive in this case and project to the lower hemisphere.

(4) For non-overhanging excavated faces, change to lower hemisphere projection all intersection vectors which project outside the area of the excavated face and containing F.

For overhanging excavated faces, change to lower hemisphere projection all intersection vectors which project inside the area of the stereogram bounded by the great circle of the excavated face and containing F. (in the cases of both non-overhanging and overhanging faces the three points resulting from the transformation are the projections of the intersection vectors in a positive direction.

(5) Test 1: If all positive intersection vectors project to the upper hemisphere, contact is maintained on three planes and the wedge is recumbent.

(6) Test 2: If the azimuths of the three positive intersection vectors do not lie within 180° of arc, then contact is lost on all planes and the wedge falls vertically under gravity loading.

(7) The positive intersection vector projecting to the lower hemisphere and having the steepest dip is determined and designated $I_{xy}$.

(8) Lines are drawn from the net centre to $I_{xy}$, $I_{xz}$, X and Y so that their relative azimuths may be readily described.

(9) Test 3: Determine and note the vector with the intermediate azimuth in each of the following cases:
(a) $I_{xy}$, X and Y  
(b) $I_{xy}$, X and $I_{xz}$  
(c) $I_{xy}$, Y and $I_{yz}$

The plane or planes on which contact is maintained are read from Table 4.6. In this table twelve possible outcomes of 3 (a), 3 (b) and 3 (c) are listed. The above method does not consider the influence of in situ stresses on the wedge but it can be a useful tool for design of support for wedges and rock joints.

**Table 4.6** Modes of sliding as determined from test 3 (after Lucas, 1980)

<table>
<thead>
<tr>
<th>No.</th>
<th>Result of test 3 (a)</th>
<th>Result of test 3 (b)</th>
<th>Result of test 3 (c)</th>
<th>Planes on which contact is determined</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>X</td>
<td>$I_{xy}$</td>
<td>$I_{xy}$</td>
<td>XY</td>
<td>Plane Y overlies Plane X</td>
</tr>
<tr>
<td>2</td>
<td>$I_{xy}$</td>
<td>$I_{xy}$</td>
<td>$I_{xy}$</td>
<td>XY</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>Y</td>
<td>$I_{xy}$</td>
<td>$I_{xy}$</td>
<td>XY</td>
<td>Plane X overlies Plane Y</td>
</tr>
<tr>
<td>4</td>
<td>X</td>
<td>Y</td>
<td>$I_{xy}$</td>
<td>X</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>$I_{xy}$</td>
<td>X</td>
<td>$I_{xy}$</td>
<td>Plane Y overlies Plane X</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Y</td>
<td>X</td>
<td>$I_{xy}$</td>
<td>Z or none*</td>
<td>See footnote</td>
</tr>
<tr>
<td>7</td>
<td>X</td>
<td>$I_{xy}$</td>
<td>Y</td>
<td>Z or none*</td>
<td>See footnote</td>
</tr>
<tr>
<td>8</td>
<td>$I_{xy}$</td>
<td>$I_{xy}$</td>
<td>Y</td>
<td>Plane X overlies Plane Y</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Y</td>
<td>$I_{xy}$</td>
<td>Y</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>X</td>
<td>X</td>
<td>Y</td>
<td>Plane X overlies Plane Y</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>$I_{xy}$</td>
<td>X</td>
<td>Y</td>
<td>Z or none*</td>
<td>See footnote</td>
</tr>
<tr>
<td>12</td>
<td>Y</td>
<td>X</td>
<td>Y</td>
<td>X</td>
<td>Plane Y overlies Plane X</td>
</tr>
</tbody>
</table>

* If the result of test 3 indicates that contact is lost on all planes, then "none", otherwise contact is maintained on Plane Z only.

**4.2.6 Determination of wedge size**

The stereographic method can be used for estimation of the size and shape of potentially unstable wedges in the roof and sidewalls of an excavation (Singh et al., 1982). In roof failure analysis three planes which have been determined by the plot of pole concentrations as the planes which form an unstable wedge are shown in Figure 4.13 as three circles 'A', 'B' and 'C'. The strike lines of these planes are marked 'a', 'b' and 'c' and the traces of the vertical planes through the centre of the net and the great circle intersections are marked 'ab', 'ac' and 'bc'.
The width of the excavation is considered to be \( W \) and the direction of the strike lines are assumed to be the traces of the planes 'A', 'B' and 'C' in the back of the excavation. The back is considered to be horizontal. With a combination of these strike lines the maximum size of the wedge which has been formed in the back of the opening can be given. The apex of the wedge can be found by the intersection of the lines ab, ac and be projected from the corners of the triangular wedge base as shown in Figure 4.13. The height of the wedge is found by taking a section through the wedge apex normal to the back of the excavation. The apparent dips of planes 'A' and 'C' are the angles \( \theta \) and \( \pi \) which are measured on the stereographic projection along the line X-X through the centre of the net. After finding the shape of the base of the wedge its area can be calculated. The volume of the wedge is given by \( \frac{1}{3} \) rd the base area \( \times \) (height), and if the density of the rock is known the mass of the wedge can be calculated.

After identifying the size and geometry of potentially unstable wedges a bolting pattern for supporting it should be designed. The spacing of the bolts should be sufficiently close to each other so that at least one bolt intersects every wedge. If the wedge has the potential to fall by gravity, the length of anchorage beyond the wedge should be determined so that it can sustain the load of the largest wedge which may be created in that area. This method is valuable in determining bolting patterns because the density and length of bolts are based on the wedge geometry. If a bolt pattern is predicted which has a density greater than one
bolt per square metre, then use of the mesh for holding small wedges in place is recommended.

In the case where a wedge can fail by sliding, the frictional resistance of the sliding surface should be taken into account when designing the support system (Heck and Brown, 1980). When the shear strength of discontinuity is defined by Coulomb's law, it can be shown that the tension required to support the wedge is given by the following equation:

\[
T = \frac{W \cdot \sin (\theta - \phi) - C \cdot A \cdot \cos \theta}{\cos (\beta - \phi)} \quad \text{Eq. (4.5)}
\]

Where:
- \( T \) = Required bolt tension to stabilise the block from sliding
- \( W \) = Weight of sliding wedge
- \( \theta \) = Angle of inclination of discontinuity to horizontal
- \( A \) = Area of contact of sliding surface of wedge
- \( \phi \) = Internal angle of friction of discontinuity
- \( C \) = Cohesion
- \( \beta \) = Angle between rockbolt and the plane of discontinuity

4.3 Rock testing techniques

Properties of rocks are divided into two main groups which are physical and mechanical properties. The main physical properties are: mineralogical composition, specific gravity, porosity, degree of saturation, permeability, chemical effects, thermal properties and electrical properties. On the other hand, for engineering applications there are three basic mechanical properties: Elastic modulus, Poisson's ratio and strength. Both physical and mechanical properties are obtained by test methods in the field or in the laboratory. Determination of mechanical properties of rocks requires a large number of properly prepared and shaped specimens.

Uniaxial and triaxial compressive tests, point load and direct shear box tests are the most important tests which will be discussed in this section. The objective of the testing program was to define the characteristic strength and deformation behaviour of the rocks. During the tests the basic properties of the rock; which are Elastic modulus, Poisson's ratio, strength, cohesion and joints friction angle were measured.
4.3.1 **Uniaxial Compressive Strength**

Uniaxial compressive strength ($\sigma_c$): The greatest compressive stress that a specimen can withstand when subjected to stress in a single direction, usually in an axial direction in the case of a cylindrical specimen. In a uniaxial compressive test a specimen of suitable geometry is loaded until failure (and post-failure if the machine is stiff). The stress at failure (the uniaxial compressive strength) can be calculated with knowledge of the load at failure and the cross-sectional area of the specimen.

Uniaxial compressive strength has been used for classification of intact rock and rock masses. An example for engineering classifications of rock on the basis of this parameter was done by Deere and Miller (1966) is shown in Table 4.7.

### Table 4.7 Example of engineering classifications of rocks based on strength (after Deere and Miller, 1966)

<table>
<thead>
<tr>
<th>Class</th>
<th>Description</th>
<th>Uniaxial Compressive Strength, MPa</th>
<th>Rock material</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Very high strength</td>
<td>$&gt; 220$</td>
<td>Majority of igneous rocks, strong metamorphic rocks: quartzite, diabase</td>
</tr>
<tr>
<td>B</td>
<td>High strength</td>
<td>110 - 220</td>
<td>Weakly cemented sandstones, hard shales</td>
</tr>
<tr>
<td>C</td>
<td>Medium strength</td>
<td>55 - 110</td>
<td>Shales, porous sandstones and limestone</td>
</tr>
<tr>
<td>D</td>
<td>Low strength</td>
<td>28 - 55</td>
<td>Porous low density rocks, tuff, clay, shales</td>
</tr>
<tr>
<td>E</td>
<td>Very low strength</td>
<td>$&lt;28$</td>
<td>Weathered and chemically altered rocks of any lithology</td>
</tr>
</tbody>
</table>

If the complete stress-strain curve is to be obtained for the specimen, enabling Elastic modulus and Poisson's ratio to be calculated, it is preferable to provide facilities for continuous recording of load, longitudinal and lateral strain. Load may be monitored by a load cell either incorporated in the machine or added externally. Strain may be monitored either by electrical resistance strain gauges or Linear Variable Differential Transducer (L.V.D.T).
Because of the heterogeneous nature of rock, the length over which the strain is measured should be as large as possible, but should not approach within $d/2$ of the ends, where $d$ is the diameter. Longitudinal and lateral strain should each be measured with a pair of active gauges mounted parallel and perpendicular to the length of the specimen. In order to prevent excessive scatter of results, application of sound experimental techniques is essential. The factors which influence results and therefore, determine sound practice may be classified as follows:

(i) Specimen preparation techniques
(ii) Tolerance of specimen geometry
(iii) Loading rate
(iv) Moisture content
(v) Loading machine characteristics

(a) Specimen preparation technique

Cylindrical specimens are considered to be the easiest to prepare, by coring either in the laboratory from bulk sample (generally 54 mm (NX) diameter) or on site and then dressing the ends to form a cylinder. The geometry of the prepared specimen must be sufficiently precise to ensure that the actual stress distribution induced during testing approximates reasonably to that assumed. The techniques used in preparation depend on whether or not the rock is significantly weakened by water.

In case of rocks not significantly weakened by water, eg. limestone, sandstone, coring is most easily done with a thin-walled diamond coring bit and water flush. In order to prevent contamination of the specimen, no additive, eg. soluble oil, should be included in the flushing water. End dressing is normally done by trimming the core with a water-cooled diamond saw and then lapping the specimen to provide true flat ends.

In the case of rocks significantly weakened by water, eg. mudstone or, siltstone, core recovery with a thin-walled bit is poor whatever flushing medium is used. The rock in this category tends to be structurally weak due to the presence of bedding planes and other structural weaknesses. An alternative arrangement for coring these rocks is a double-walled core barrel together with a diamond saw-tooth bit, and also air is used for cooling. This technique is used to help preserve the structural integrity of the specimen.
(b) Specifications and tolerances for specimen geometry

Control of specimen geometry is intended to ensure that the action of the testing machine induces a predictable uniform stress in the central section of the specimen, remote from the end effects at the platens. Control of the length/diameter (L/D) ratio of the specimen is derived from experimental results similar to those given in the Figure 4.14. This Figure shows that the uniaxial compressive strength of a given rock is a function of L/D, decreasing as L/D increases due to the reduction in the proportion of the specimen restrained by the platens. With L/D greater than 2, preferably 2.5, but less than 4, consistent results can be obtained. The upper value of 4 is necessary to limit possible instabilities due to buckling. Tolerances recommended by Hawkes and Mellor (1970), and by the International Society of Rock Mechanics (1981) are given in Table 4.8.

(c) Loading rate

Load on the specimen should be applied continuously at a constant stress rate such that failure occurs within 5-10 min of loading. The ISRM (1981) specify a rate of 1-0.5 MPa/s for stress rate controlled tests and 1 mm/s for displacement controlled tests.

(d) Moisture content

Moisture can have a significant effect on the deformability of the test specimen. When possible, in situ moisture condition should be preserved until the time of test. While the properties of well-cemented or compacted rocks such as limestone, sandstone and slate are
little affected by moisture content, considerable reduction can occur in the strength of sedimentary rocks such as mudstone, siltstone and shales as moisture content increases. It is important therefore, to standardise moisture content during testing.

Table 4.8 Tolerances recommended by the ISRM (1981) for sample preparation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specification or tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length/Diameter ratio</td>
<td>2.5 &lt; L/D &lt; 3</td>
</tr>
<tr>
<td>Flattness of ends</td>
<td>flat to within 0.02 mm</td>
</tr>
<tr>
<td>Squareness of ends</td>
<td>to within 0.001 radians</td>
</tr>
<tr>
<td>Generators of cylindrical surfaces</td>
<td>straight to within 0.3 mm</td>
</tr>
<tr>
<td></td>
<td>specimen should be smooth and free from irregularities</td>
</tr>
<tr>
<td>Diameter</td>
<td>should be measured to the nearest 0.1 mm</td>
</tr>
</tbody>
</table>

(e) Test procedure and loading machine characteristics

An INSTRON 8033 servo-controlled testing machine was used in these tests. This machine is capable of applying compressive and tensile loads of 500 KN over a working stroke from -75 mm to +75 mm with an overall system stiffness greater than 1060 KN/mm. Figure 4.15 shows details of the servo controlled testing machine which has been used for uniaxial and triaxial tests.

The specimen, which has been prepared according to the recommendations in Figure 4.14 and Table 4.8, is mounted centrally on the testing machine between steel platen of the same diameter. Load is applied at a constant rate and if uniaxial compressive strength alone is required, the total load at which the specimen fails is noted. Knowing the cross-sectional area of the specimen, the stress \( \sigma_c \) at failure can be obtained. Since it is difficult to ensure the upper and lower end surfaces of the specimen are parallel, a spherical seat is usually inserted between the lower end surface of the specimen and the lower machine platen. This allows complete contact and therefore, a uniform loading at the time of testing. The corresponding strains in the specimen are recorded by mounting strain gauges on the surface of the specimen, both parallel and perpendicular to the direction of applied load.
Figure 4.15 The servo controlled testing machine used for uniaxial and triaxial tests
The uniaxial compressive strength of the specimen is obtained by dividing the maximum load carried by the specimen during the test by the original cross sectional area.

\[ \sigma = \frac{P}{A} \]  
Eq. (4.6)

The stiffness found from the following:

\[ K = \frac{P}{\Delta L} \]  
Eq. (4.7)

where:

- \( \sigma \) = Stress, MPa
- \( P \) = Normal load, N
- \( A \) = Area, mm²
- \( K \) = Stiffness of the material, N/mm
- \( L \) = Length of the specimen, mm
- \( \Delta L \) = Change of the length of the specimen, mm

The axial and diametric strain are recorded directly from the strainmeter which is connected to the strain gauges. The strains can also be calculated by the following equations:

\[ \varepsilon_a = \frac{\Delta L}{L} \]  
Eq. (4.8)

\[ \varepsilon_d = \frac{\Delta d}{d} \]  
Eq. (4.9)

where:

- \( \varepsilon_a \) = Axial strain
- \( \varepsilon_d \) = Diametric strain
- \( L \) = Original measured axial length
- \( \Delta L \) = Change in measured axial length (defined to be positive for a decrease in length).

Elastic modulus (E) is the ratio of normal stress to strain for a material at a specified stress level when subjected to stress in a single direction. It should be mentioned that the stress-strain curve is rarely linear for rocks and such various methods of determining elastic modulus are used:
1. Tangent modulus, $E_t$, is obtained at a stress level which is a fixed percentage (usually 50%) of the ultimate uniaxial compressive strength.

2. Average modulus, $E_{av}$, is the average slope of straight line portion of the stress-strain curve.

3. Secant modulus, $E_s$, is usually measured from zero stress to some fixed percentage of the ultimate strength, generally at 50%.

In this thesis the tangent modulus at 50% is used.

Poisson's ratio ($v$) is the ratio between lateral strain and axial strain) strain of the specimen subject to uniaxial stress. Poisson's ratio ($v$) is therefore, calculated in the following manner:

$$v = \frac{\text{Lateral Strain}}{\text{Axial Strain}}$$

The larger the value of $E$ and $K$, the stiffer the material is. For coal, $E$ usually falls between 0.5 and 5 GPa, whereas for rocks it is 5 to 70 GPa. The larger the value of $v$, the more expandable the material is. This value generally ranges from 0.06 to 0.45. According to the theory of elasticity, the maximum value for $v$ is 0.5.

For a given stress-strain level the volumetric strain is calculated from the following equation:

$$\varepsilon_v = \varepsilon_a + \varepsilon_d$$ \hspace{1cm} \text{Eq. (4.10)}$$

### 4.3.2 Point Load Test

The point load test is used to estimate indirectly the uniaxial compressive strength of the rocks by measuring the point load strength as an index ($Is(50)$) (Brock and Franklin, 1972). The test can also measure the strength anisotropy index ($Ia(50)$) which is the ratio of point load strengths in the direction which gives the greatest and least values. The specimens can have any shape and size. The ideal form of the specimen is the NX size core with a length of at least 1.5 times the diameter. The core fails at a relatively low applied force ($P$) due to the tensile stresses over the diametrical area between the points.
(a) Testing techniques

Two forms of test can be carried out. These are the diametric and the axial tests with saw cut faces. Specimens suitable for using in diametric test are cylindrical (minimum diameter of NX core size = 54 mm) with a length to diameter ratio of 1.5. In the diametric point load test the failure load 'P' is independent of the length of the core assuming that the distance L is sufficiently large. Core specimens with a length to diameter ratio around 1.5 are used in these tests.

![Diametric point load test](image)

**Figure 4.16** Diametric point load test

In the axial point load test the length and diameter of each specimen influences the results. In this test a core specimen with a length to diameter ratio of around 1 is used and the load is applied to the axis of the core specimen.

(b) Test procedure

The testing machine consists of a loading system comprising a loading frame, pump, ram, cone platens, a system for measuring the load 'P' for breaking the specimen and a system for measuring the distance 'D' between the two platen contact points. The platens remain co-axial during the tests. The geometry of the cones is standardised at an angle of 60° and a radius of curvature of 5 mm.

Load is measured by monitoring the hydraulic pressure in the jack. A maximum pressure indicating needle is needed because it is very difficult to read the failure load reliably.

The distance measuring system, which is a metal scale calibrated in millimetres, should be fixed to a cross-head of the testing machine. One of the pointers is located on the lower platen in such a way as to allow measurement of the platen separation irrespective of cross-head position or ram travel. The distance 'D' used in calculating the strength index, is
defined as the distance between the platen points at the moment of failure, and it is equal to the diameter of the specimen at the start of the test only if the specimen is hard and the platens do not penetrate it. For hard rocks an initial reading of 'D' is sufficiently accurate.

The point load index (Is) indicates the strength at failure and is given by the following equation:

\[ I_s = \frac{P}{D^2} \]  

Eq. (4.11)

where:

\( P \) = The load required to break the specimen, KN
\( D \) = The diameter of the rock specimen, mm

This index has a very close correlation with uniaxial compressive strength. This relationship is shown by the following equation:

\[ \sigma_c = C \cdot I_s \]  

Eq. (4.12)

Where:

\( \sigma_c \) = The uniaxial compressive strength, MPa
\( I_s \) = The point load strength index, MPa

The value 'C' is a constant equal to 24 for NX (54 mm diameter) cores. Other values suggested for 'C' are as follows:

<table>
<thead>
<tr>
<th>Core diameter (mm)</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>17.5</td>
</tr>
<tr>
<td>30</td>
<td>19</td>
</tr>
<tr>
<td>40</td>
<td>21</td>
</tr>
<tr>
<td>50</td>
<td>23</td>
</tr>
<tr>
<td>60</td>
<td>24.5</td>
</tr>
</tbody>
</table>

The test is valid only if a clean diametrical break occurs between cores. If the fracture runs to another plane or if there are signs of core penetration and crushing as in weaker rocks, the results should be rejected.
Since \( I_s \) varies as a function of 'D', a size correction value should be applied to obtain a standardised point load strength for any size of core. The standard or size corrected value is (Brock and Franklin, 1972):

\[
I_s(50) = \frac{F \cdot P}{D^2}
\]

Eq. (4.13)

where:

\[
F = \left( \frac{D}{50} \right)^{0.45}
\]

Eq. (4.14)

For tests near the standard 50 mm core size, this equation can be written as follows with enough accuracy:

\[
F = \left( \frac{D}{50} \right)^{0.5}
\]

Eq. (4.15)

4.3.3 Triaxial Test

Triaxial strength is the greatest compressive stress that a specimen can maintain in the major principal stress direction when subjected to a confining minor and/or intermediate stress. The triaxial test is used to determine the stress deformation characteristics of cylindrical specimen in a triaxial stress state. This provides the values necessary to determine the strength envelope and from this the value of the internal friction angle and cohesion may be calculated.

The aim of the triaxial test is to simulate the conditions which may occur in the rock material around an excavation or under a foundation where the rock is subjected to confining pressures and deviatoric stresses. However, a full series of triaxial compressive tests is only very rarely required since methods for estimating the complete shear strength to normal stress relationship are available and such estimates are usually adequate for practical engineering purposes. Triaxial compression tests are usually performed when a confirmation of the estimated failure criteria is required. The cylindrical specimens obtained from core drilling are used for triaxial testing. The specimens are located in a rubber membrane, which in turn is located in the triaxial cell which is essentially a cylindrical steel chamber large enough to accommodate a specimen with a length to diameter ratio of more than 2. A hydraulic pump is used to provide the confining pressure.

(a) Sample preparation

Cylindrical specimens obtained from core drilling are used for triaxial testing. Sample preparation is similar to the uniaxial test but in this test the specimen should not be stored more than 30 days before testing. The specimen should have a length to diameter ratio of
between 2.0 and 3.0 and a diameter of approximately 54.0 mm (NX size). In order to preserve the natural water content of the specimens they should be covered with cling film. At least three specimens of the same rock should be tested at different confining pressures.

(b) Test procedure

The triaxial cell used in this investigation is commercially available as shown in Figure 4.17. After securing the specimen inside the cell as shown in Figure 4.17 the axial load is applied in the same way as the uniaxial compressive test. The confining pressure is applied by using hydraulic oil and a hydraulic pump to maintain constant lateral pressure. Similar to the uniaxial compressive test, a spherical seat is inserted between the specimen and the upper and lower ends of the triaxial cell for applying uniform load on the specimen. The hydraulic oil is pumped into the cell until the predetermined confining pressure is reached. At this stage, the specimen is subjected to a uniform load in all directions. The vertical load is then gradually increased through the loading piston until failure. The maximum axial load at failure ($\sigma_1$) and the corresponding confining pressure ($\sigma_3$) is recorded. From these values Mohr's stress circle can be constructed.

![Diagram of triaxial cell](image)

**Figure 4.17** Section view of the triaxial cell
The confining compressive strength of the specimen is obtained by dividing the maximum axial load applied to the specimen during the test, by the original cross sectional area. The apparent cohesion, $C$, and internal friction angle, $\phi$, are calculated from the following formulas:

\[ \phi = \arcsin \left( \frac{m - 1}{m + 1} \right) \]  
Eq.(4.16)

\[ C = b \left( \frac{1 - \sin \phi}{2 \cos \phi} \right) \]  
Eq.(4.17)

Where:

- $m = \text{The slope of the straight line portion of the } \sigma_1 \text{ versus } \sigma_3 \text{ curve.}$
- $b = \text{The intercept of the straight line portion with the } \sigma_1 \text{ axis.}$

Alternatively by repeating this test three times with different values of confining stress the Mohr's envelope can be constructed. Figure 4.18 shows the method of construction of Mohr's envelope from the results of triaxial tests. The rock would be stable when the state of the stress is below the Mohr's envelope and it would be unstable when the state of the stress is above the envelope.

![Figure 4.18 Construction of Mohr's envelope.](image)
4.3.4 Direct shear test

This test is used to investigate the shear strength of intact soft rock, plane of weakness and frictional properties of discontinuities. From this test the peak and residual shear strength as a function of the normal stress to the sheared plane can be measured. The inclination of the test specimen with respect to the rock mass, and its direction of placement it in the testing machine, are usually selected so that the shear plane coincides with a plane of weakness in the rock, for example a joint, plane of bedding, schistosity or cleavage. The shear strength is determined by conducting a series of tests on the same horizon with each specimen tested at different but constant normal stress.

(a) Shear rate and ultimate shear displacement

The ISRM (1981) has recommended a shear displacement rate of 0.1 mm/min for shear tests on rock joints and granular materials. The ultimate shear displacement during a shear test is a function of the mechanical configuration and size of the shear box. Shear displacement of about 10 mm are usually sufficient to give values of residual shear strength. This should also be noted that in order to preserve near axial symmetry of normal loading, shearing should not be carried for more than 0 mm from the starting position in either direction (Potable Shear Box catalogue).

(b) Test equipment

The direct shear test is a method for determination of the shear strength of rock joint. The equipment is a portable Hoek Shear Box (Figure 4.19) which can contain a maximum rock size of 115 mm x 125 mm or core of up to 102 mm diameter. It consists of two halves, the lower part is fitted with two rams for applying the reversible shear force, and the upper part is fitted with a ram for normal load application. The means for applying the normal force is a hydraulic system with a hand pump which is designed in a way to ensure that the load is uniformly distributed over the plane to be tested. The resultant force acts normal to the shear plane passing through its centre of area.

The means for applying the shear force is also a hydraulic jack designed so that the load is uniformly distributed along one half-face of the specimen. The resultant applied shear force will act on the plane of shearing. There are two gauges for independent measurements of the applied shear and normal forces and a scale for reading up to 50 KN calibrated to 1 KN. These two gauges should be calibrated before starting the tests. The equipment for measuring shear and normal displacements are micrometer dial gauges with an accuracy of
$10^{-2}$ mm. Clamp attachments were provided so that the dial gauge could be mounted to record horizontal movement of the box.

(c) Sample preparation

A sample containing the discontinuity or plane of weakness is selected from the cores and is trimmed so that it can be fitted into a mould. The two halves of the specimen should be wound together in order to prevent movement along the discontinuity and the sample then should be placed in the clamp to be ready for mounting in the mould. A quick setting plaster/mortar should be prepared for use as the bonding medium. The mould is coated with mould release oil in order to facilitate sample removal on setting of the mounting medium. The clamp is placed with the wire bound specimen gripped in its jaws across the half mould which contains the retaining screws in its inclined faces. The shear plane should be placed in the horizontal plane so that an even normal force can be applied on it. The plaster should be allowed to become hard so that the cast supporting bolts don’t break out of the bounding when turning the mould upside down (Figure 4.20). After casting the sample and removing it from the mould it should be left for a few days so that the plaster becomes sufficiently hard and suitable for testing.

(d) Test procedure

After placing the sample inside the shear box a small force is applied to make sure that the joint surface has been held uniformly. The specimen is then ready for testing. The normal load is then raised to reach the value specified for the test. This normal load should be held
constant while the shear load increases. The direction of the shear force is parallel to the contact plane between two pieces and is gradually increased until failure occurs along the contact plane. The measured shear strength is proportional to this normal force. From the maximum load the shear strength ($\tau_f$) is calculated. Then the specimen is sheared until a constant shear load is reached which corresponds to the residual strength ($\tau_r$).

Figure 4.20 Typical arrangement of direct shear test on a joint.

The horizontal force at failure divided by the cross sectional area of the sheared area is the shear strength. The normal and shear stresses are computed as follows:

\[
\text{Normal stress} \quad \sigma_n = \frac{P_n}{A} \quad \text{Eq. (4.18)}
\]
\[
\text{Shear stress} \quad \tau = \frac{P_s}{A} \quad \text{Eq. (4.19)}
\]

Where:
\[
P_n \quad = \text{Normal force, MN}
\]
\[
P_s \quad = \text{Maximum shear force at failure, MPa}
\]
\[
A \quad = \text{Area of shear surface, mm}^2
\]

Graphs of shear stress (or shear force) versus shear displacement are plotted. Values of the peak and residual shear strength and also shear displacements are obtained from these graphs. Graphs of peak and residual shear strength versus normal stress are plotted from the combined results for all the test specimens. By repeating the above test (at least three times) with different values of normal stress ($\sigma_n$) the relations between shear strength
(τ_p), residual shear strength (τ_r) and (σ_n) can be determined. These graphs are approximately a straight line (Coulomb's straight line shear strength relationship) as shown in Figure 4.21.

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

**Figure 4.21** Relationship between shear strength and normal stress at failure.

The Coulomb criteria assumes that shear failure of a material which is loaded in compression is induced if the shear stress acting along the potential shear failure planes is equal to:

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

\[ \tan \phi = \mu \]

Eq. (4.21)

Where:

- \( \phi \) = Angle of friction, degree
- \( \tan \phi \) = Coefficient of friction of the shear surface
- \( C \) = Apparent cohesion, MPa

4.4 Conclusions:

Field investigation supplemented by rock testing enables the acquisition of accurate information necessary for stability assessment of crown pillar. Joint sets are the most important structural weakness which should be distinguished in field investigation. Potentially unstable wedges can be formed in underground excavations by the intersection of different joint sets identified by joint surveying. By using the method of stereographic prediction of wedge failure suggested by Lucas (1980) potentially unstable wedges can be identified and supported.
Laboratory testing techniques enable the determination of deformation properties of rocks necessary for engineering design eg. tangent modulus, Poisson's ratio and shear strength of rock joints. It should be recognised that the level of confidence in values obtained from testing rock samples can be influenced by the test method, testing instruments and therefore, insitu tests where justified by cost and time involved usually give more valid results.
CHAPTER 5

INVESTIGATION INTO THE STABILITY OF CROWN PILLARS IN A COPPER MINE - A CASE STUDY

5.1 Introduction

The CSA Mine, Cobar, NSW, Australia was chosen as a site for stability evaluation of crown pillars. Cobar is a copper mine where open stoping operations are carried out in several steeply dipping, parallel orebodies. The Cobar mining field occurs within the deep marine Early Devonian Cobar Trough, or Basin, which is considered to be one of many meridional grabens that developed in the Lachlan Fold Belt during the Siluro-Devonian era (Cobar Mines, 1990). The deposits of the Cobar field are generally regarded as the belt of mineralisation that extends from the Queen Bee Mine in the south to Elura in the north (Figure 5.1) and occurs only within the Nurri and Amphitheatre Groups of the Cobar Supergroup. The only member of the Amphitheatre Group which is economically significant in the Cobar area is the CSA Siltstone which consists of a thinly bedded turbiditic sequence of carbonaceous siltstone and mudstones with minor thin, fine to medium-grained sandstones. Thick, massive sandstone beds occur locally. Bedding is generally graded and a variety of sedimentary structures can be observed.
Figure 5.1 Regional geology of CSA Mine Cobar

The CSA copper-lead-zinc orebodies occur within steeply dipping north-south trending shear zones which cut across the sedimentary rocks of the Upper Silurian-Lower Devonian Cobar group (Cobar Mines, 1990). The CSA mine ore bodies are located within quartz rich shear zones which strike north-south and dip 75° to 85° to the east. The bedding planes strike at approximately 345° and dip at 80° whilst the dominant cleavage and shear zones dip east parallel to the orebodies (Figure 5.2).
A broad halo of pervasive chlorite alteration surrounds individual lenses for up to 50 metres laterally. In many cases this is accompanied by a pervasive silicification. The chlorite alteration is predominantly green in colour and may only occur as a coating on cleavage surfaces in the outermost part of the halo. A magnesium-rich black chlorite is associated with certain ore types, as is talc. Later shears containing black chlorite and
occasionally talc are common in all mineralised areas and in some cases form a sharp boundary to the ore lenses.

The country rock is composed of chloritic and quartzitic siltstone. The country rocks and ore are cut by shear zones in which chloritic association are common (Barton, 1978). The shear zones are extensive and located mostly along footwalls of the orebody. Macroscopic faults occur throughout the mine area, but are more in the broken zones which surround the orebody. The majority of faults are subparallel to the cleavage but others are extended subparallel to the bedding or normal to the main cleavage and orebody. The joints are discontinuous with wavy and rough surfaces and have a spacing ranging from 0.3 to 1 m (an average spacing of 0.5 m).

The insitu virgin stress field as presented by Worotniki et al (1975) and Maconochie et al (1981) is shown in table 5.1 and Figure 5.3. All normal stress components at Cobar increase with depth below surface, the horizontal stress is greater than the vertical and the east-west stress is greater than the north-south stress. The major principal stress is approximately normal to the plane of the orebodies.

**Table 5.1** Virgin insitu field stress at CSA Mine, (after Maconochie et al, 1981)

<table>
<thead>
<tr>
<th>Normal stress</th>
<th>Relative value</th>
<th>Value at 550 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>West</td>
<td>( \sigma_1 = 10 + 0.033Z )</td>
<td>28.7, MPa</td>
</tr>
<tr>
<td>North</td>
<td>( \sigma_2 = 2 + 0.030Z )</td>
<td>18.8, MPa</td>
</tr>
<tr>
<td>Vertical</td>
<td>( \sigma_v = 0 + 0.029Z )</td>
<td>16, MPa</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shear stress</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Direction</td>
<td>Relative value</td>
<td>Value at 550 m</td>
</tr>
<tr>
<td>Vertical/West</td>
<td>( \tau_w = 0 + 0.012Z )</td>
<td>4.32, MPa</td>
</tr>
<tr>
<td>Vertical/North</td>
<td>( \tau_n = 0 )</td>
<td>0, MPa</td>
</tr>
<tr>
<td>North/West</td>
<td>( \tau_{nw} = 0 )</td>
<td>0, MPa</td>
</tr>
</tbody>
</table>
Figure 5.3 Virgin field stress in CSA Mine, Cobar (after Worotnicki, et al, 1975)

CSA Mine produces copper, lead and zinc concentrates for sale in Australia and overseas. Mining is by conventional and retreat long-hole open stoping with hydraulic and mullrock back-fill. Stopes are generally 30-40 m high, 30-60 m long and 5-20 m wide. The orebody has 2-3% copper, 2-3% zinc, 0.5-1% lead and 20-25 g/t silver. Ore from stoping operations is transferred by truck and loader to a primary crusher on 9 level (810 m below surface) and then by skip to the surface. The ore is stored in crushed ore bins, milled and by flotation the concentrates of copper, lead and zinc are obtained respectively. The current production rate is 900,000 tonnes per annum and reserves at the end of 1989 were 5 million tonnes with 2.92% Cu, 0.55% Pb and 1.49% Zn. Figures 5.4 and 5.5 show the conventional and retreat long-hole open stoping method use at CSA.
Figure 5.4 Conventional longhole open stoping method

Figure 5.5 Retreat longhole open stoping method
5.2 Crown pillars design method in CSA Mine, Cobar

The crown pillar in tabular orebodies like CSA Mine, Cobar, for most cases are rectangular and have relatively simple geometry. Most of the crown pillars had been designed using past experience of the mine engineers or rule of thumb supplemented by numerical methods for stress analysis. A typical geometry of a crown pillar in CSA Mine is illustrated in Figure 5.6.

Most pillars were stable and utilised cable bolts for reinforcement. Generally in the area under study, the width of the crown pillars was equal to the width of orebody, 12 m on average, and had a thickness of 15 metres. Crown pillars in this mine are rectangular and have a relatively simple geometry. They are supported by 6 to 8 cable bolts with a length of 6 to 10 m. A typical support system for crown pillars in this mine is illustrated in Figure 5.7.

![Figure 5.6 Typical geometry of a crown pillar in CSA Mine.](image)
Chapter 5, Investigation into the stability of crown pillars in a copper mine - a case study

Observations have shown that most crown pillars in CSA Mine are stable by use of cable bolts. However, some minor and major crown pillar problems have been experienced both during and after the excavation of stopes. Generally, information about the behaviour of the pillar during failure was not available. These failures can be related to the structural weakness, backfilling the upper stope, stoping development in adjacent areas and concentration of high stresses in particular points of the crown pillar or stope walls. Crown pillar failure has often been attributed to flat dipping shear zones containing black chloritic schist.

5.3 Field study

Data collection was carried out in the mine to obtain information about the regional and local mine geology, virgin field stress and method of design and support of crown pillars in particular. A joint survey was also performed to help assess the stability status of the underground excavation and to assist in obtaining an initial understanding of the structure of the rock mass. The results of the joint survey were used for analysing the potential for individual wedge failure.

The method of joint surveying composed measurements along three scanlines, each approximately 10 m in length, two of which were horizontal, the other vertical. The data were collected by recording details of the discontinuities intersecting the scanline. A geological magnetic compass was used for the field measurements. Figure 5.8 shows the plan of the area in which the joint surveying was carried out.

Figure 5.7 Typical support for crown pillars in CSA Mine, Cobar.
The discontinuities orientations were plotted on a Schmidt equal area lower hemisphere stereonet. Figure 5.9 is scatter plot of pole concentrations at the site of investigation. Figure 5.10 and 5.11 show the contour diagram and planes of the major joint sets for this area. Two near vertical joint sets and random flat dipping joints are the most important structural weakness which were distinguished in this investigation. The dip of the horizontal random joint set is about 15 degrees and the vertical joint sets have an average dip of 70-80 degree but with opposite dip direction.

Figure 5.8 Plan view of the area of joint surveying at the site of investigation.
Figure 5.9 Scatter plot of poles concentration at the site of investigation.
Figure 5.10 Contour diagram of poles at the site of investigation.
5.3.1 Determination of the possible mode of failure of rock wedges

Potentially unstable wedges have been formed by the intersection of the three joint sets identified by the survey. Using the suggested method of stereographic prediction of wedge failure by Lucas (1980), described in Section 4.2.5, the possible mode of failure of this wedge is discussed here. The dip/azimuth of the planes forming the wedge are:

<table>
<thead>
<tr>
<th>joint set</th>
<th>Plane A</th>
<th>84 /276</th>
</tr>
</thead>
<tbody>
<tr>
<td>joint set</td>
<td>Plane B</td>
<td>83 /074</td>
</tr>
<tr>
<td>joint set</td>
<td>Plane C</td>
<td>22 /091</td>
</tr>
<tr>
<td>stope roof</td>
<td>Plane F</td>
<td>00 /-</td>
</tr>
</tbody>
</table>

Figure 5.11 Planes of major joint sets.
The positive intersection vectors $I_{ab}$, $I_{bc}$ and $I_{ac}$ all projected to the lower hemisphere (Figure 5.11). $I_{ab}$ has the steepest dip and it was assigned as $I_{xy}$. The following assignment was done for planes and intersections:

\[
\begin{align*}
I_{xy} &= I_{ab} \\
I_{xz} &= I_{ac} \\
I_{yz} &= I_{bc} \\
X &= A \\
Y &= B
\end{align*}
\]

Because the azimuths of the three intersection vectors are within 180° of arc the wedge can not fall vertically and for determination of the possible mode of failure test 3 should be used. The result of test 3 as described in section 4.2.5 has been shown in Table 5.2

**Table 5.2** The results of test 3 for determination of mode of failure

<table>
<thead>
<tr>
<th>Test 3</th>
<th>Comparisons of azimuths</th>
<th>Intermediate azimuth</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>$I_{xy}$, X and Y</td>
<td>$I_{xy}$</td>
<td>$I_{xy} \equiv I_{ab}$</td>
</tr>
<tr>
<td>b</td>
<td>$I_{xy}$, X and $I_{xz}$</td>
<td>$I_{xz}$</td>
<td>$I_{xy} \equiv I_{ac}$</td>
</tr>
<tr>
<td>c</td>
<td>$I_{xy}$, Y and $I_{yz}$</td>
<td>$I_{xy}$</td>
<td>$I_{xy} \equiv I_{ab}$</td>
</tr>
</tbody>
</table>

From table 4.6 it can be concluded that for the wedge which has been formed in the crown pillar, contact is lost on all planes or contact is maintained on the Z plane only.

### 5.3.2 Results

Bedding planes, cleavages and random flat dipper joints are the most important structural weaknesses distinguished in this investigation. The dip of the horizontal random joint set is about 15 degree and the vertical joint sets have an average dip of 70-80 degree but with opposite dip directions. Potentially unstable wedges are formed by the three joint sets identified by the survey. The results of the analysis show that contact is lost on all planes or contact is maintained on the Z plane only. This indicates that after excavation of the stopes, wedges will be formed in the crown pillar and support is required to prevent individual wedge failure which may lead to progressive failure in the crown pillar.
5.4 Rock properties and laboratory rock testing

Information on the mechanical properties of the mine rocks can be obtained from various sources. The most important sources are laboratory tests which were carried out by Barton (1978), Worotnicki (1982) and the CSIRO Geomechanics Laboratory. A series of tests were also carried out in Rock Mechanics Laboratory, University of Wollongong. The mechanical properties of rocks have been derived from the results of these laboratory tests.

(a) Uniaxial test

The results of this test on high grade ore and siltstone (country rock) are presented in Tables 5.3 and 5.4.

<table>
<thead>
<tr>
<th>Table 5.3</th>
<th>Data obtained from uniaxial tests on siltstone (country rock)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test No.</td>
<td>$\sigma_c$, MPa</td>
</tr>
<tr>
<td>1</td>
<td>98</td>
</tr>
<tr>
<td>2</td>
<td>98</td>
</tr>
<tr>
<td>3</td>
<td>108</td>
</tr>
<tr>
<td>Average</td>
<td>101</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 5.4</th>
<th>Data obtained from uniaxial tests on high grade ore</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test No.</td>
<td>$\sigma_c$, MPa</td>
</tr>
<tr>
<td>1</td>
<td>131</td>
</tr>
<tr>
<td>2</td>
<td>163</td>
</tr>
<tr>
<td>3</td>
<td>123</td>
</tr>
<tr>
<td>Average</td>
<td>139</td>
</tr>
</tbody>
</table>

(b) Point Load test

The results of this test on high grade ore and siltstone are presented in Tables 5.5 and 5.6.
Table 5.5 Results of point load tests on high grade ore

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Diametric</th>
<th>Axial</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_c$, MPa</td>
<td>$\sigma_c$, MPa</td>
</tr>
<tr>
<td></td>
<td>127</td>
<td>82</td>
</tr>
<tr>
<td>1</td>
<td>134</td>
<td>143</td>
</tr>
<tr>
<td>2</td>
<td>96</td>
<td>111</td>
</tr>
<tr>
<td>3</td>
<td>155</td>
<td>129</td>
</tr>
<tr>
<td>4</td>
<td>130</td>
<td>114</td>
</tr>
<tr>
<td>Average</td>
<td>128</td>
<td>116</td>
</tr>
</tbody>
</table>

Table 5.6 Results of point load tests on siltstone (country rock).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Diametric</th>
<th>Axial</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_c$, MPa</td>
<td>$\sigma_c$, MPa</td>
</tr>
<tr>
<td></td>
<td>77</td>
<td>77</td>
</tr>
<tr>
<td>1</td>
<td>83</td>
<td>117</td>
</tr>
<tr>
<td>2</td>
<td>110</td>
<td>94</td>
</tr>
<tr>
<td>3</td>
<td>56</td>
<td>89</td>
</tr>
<tr>
<td>Average</td>
<td>81.5</td>
<td>94</td>
</tr>
</tbody>
</table>

(c) Triaxial test

The aim of the triaxial test is to simulate the conditions which may occur in the rock material around the excavation or under a foundation where the rock is subjected to confining pressures and deviatoric stresses. The result of this test are presented in Tables 5.7 and 5.8 and Figures 5.12 to 5.15.

Table 5.7 Data obtained from triaxial tests for country rock (siltstone)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Confining stress, MPa</th>
<th>Normal stress, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>128</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>138</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>153</td>
</tr>
</tbody>
</table>

Table 5.8 Data obtained from triaxial tests on high grade ore.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Confining stress, MPa</th>
<th>Normal stress, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>135</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>154</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>170</td>
</tr>
</tbody>
</table>
Figure 5.12 Construction of Mohr's envelope for siltstone (country rock).

Figure 5.13 variation of s1 versus s3 for siltstone (country rock).
Figure 5.14 Construction of Mohr's envelope for high grade ore.

Figure 5.15 Variation of $\sigma_1$ versus $\sigma_3$ for high grade ore.
(d) Direct Shear test

The results of this test are presented in Table 5.9 and Figures 5.16 and 5.17.

**Table 5.9** Results of direct shear test on high grade ore.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Normal load</th>
<th>( \sigma_n ), MPa</th>
<th>( \tau ), MPa</th>
<th>Displacement, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5 KN</td>
<td>2.2</td>
<td>3.9</td>
<td>1.3</td>
</tr>
<tr>
<td>2</td>
<td>9 KN</td>
<td>3.9</td>
<td>4.1</td>
<td>1.8</td>
</tr>
<tr>
<td>3</td>
<td>13 KN</td>
<td>5.67</td>
<td>6.2</td>
<td>1.9</td>
</tr>
<tr>
<td>4</td>
<td>20 KN</td>
<td>8.52</td>
<td>9.1</td>
<td>2.1</td>
</tr>
</tbody>
</table>

**Figure 5.16** Maximum shear stress versus normal stress (results of direct shear test on high grade ore)
Table 5.11 Results of direct shear test on country rock (siltstone).

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Normal load</th>
<th>$\sigma_n$, Mpa</th>
<th>$\tau$, Mpa</th>
<th>Displacement, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5 KN</td>
<td>2.2</td>
<td>3.9</td>
<td>1.3</td>
</tr>
<tr>
<td>2</td>
<td>9 KN</td>
<td>3.9</td>
<td>4.5</td>
<td>1.8</td>
</tr>
<tr>
<td>3</td>
<td>13 KN</td>
<td>5.67</td>
<td>5.6</td>
<td>1.9</td>
</tr>
</tbody>
</table>

Figure 5.17 Maximum shear stress versus normal stress (results of direct shear test on country rock)

5.4.1 General results

The properties for the rock obtained from the laboratory tests are shown in Table 5.10. These results indicate that the high grade ore is strong and the siltstone (country rock) may be classified as a moderately strong rock. Determination of elastic properties shows that the country rock has an average Modulus of 62 GPa and a Poisson’s ratio of 0.21. The Modulus of the orebody was determined to be 73 GPa and Poisson’s ratio, 0.22. These results show that the orebody is a strong rock and can tolerate high stress concentrations. Properties of rock types in CSA Mine, Cobar based on data collected from different sources (Section 5.5) are shown in table 5.11. Comparison of the results indicate that they verify each other, however because the samples have been collected from different parts of the mine, there are some differences between the values obtained for each property.
### Table 5.10 Mechanical properties of rock from laboratory tests

<table>
<thead>
<tr>
<th>Properties</th>
<th>type of test</th>
<th>country rock, siltstone</th>
<th>High grade ore</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS, MPa</td>
<td>Uniaxial test</td>
<td>101</td>
<td>139</td>
</tr>
<tr>
<td>UCS, MPa</td>
<td>Point load test</td>
<td>88</td>
<td>122</td>
</tr>
<tr>
<td>Elastic modulus, GPa</td>
<td>Uniaxial test</td>
<td>62</td>
<td>73</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>Uniaxial test</td>
<td>0.21</td>
<td>0.22</td>
</tr>
<tr>
<td>Friction angle, Degrees</td>
<td>Triaxial test, Mohr's envelope</td>
<td>41°</td>
<td>45°</td>
</tr>
<tr>
<td>Internal friction angle</td>
<td>Triaxial test, strength envelope</td>
<td>42</td>
<td>45</td>
</tr>
<tr>
<td>Apparent cohesion, MPa</td>
<td>Triaxial test, Mohr's envelope</td>
<td>34</td>
<td>50</td>
</tr>
<tr>
<td>Joint friction angle, Degrees</td>
<td>Field shear box</td>
<td>37°</td>
<td>35°</td>
</tr>
<tr>
<td>Density, kg/m³</td>
<td>Mass/Volume</td>
<td>2900</td>
<td>3500</td>
</tr>
</tbody>
</table>

UCS = Uniaxial Compressive Strength

### Table 5.11 Properties of rock types in CSA Mine, Cobar (Based on data collected from different sources)

<table>
<thead>
<tr>
<th>Properties</th>
<th>Siltstone, country rock</th>
<th>orebody</th>
<th>Black chlorotic schist</th>
<th>Shear zone material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density, kg/m³</td>
<td>2900</td>
<td>2900</td>
<td>2900</td>
<td>2900</td>
</tr>
<tr>
<td>UCS, MPa</td>
<td>65 - 120</td>
<td>116</td>
<td>46</td>
<td>NA</td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>.25</td>
<td>.25</td>
<td>.25</td>
<td>.25</td>
</tr>
<tr>
<td>Elastic modulus, Gpa</td>
<td>58</td>
<td>85</td>
<td>42</td>
<td>35</td>
</tr>
<tr>
<td>Friction angle, degree</td>
<td>38</td>
<td>38</td>
<td>38</td>
<td>NA</td>
</tr>
<tr>
<td>Tensile strength, MPa</td>
<td>15</td>
<td>15</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

UCS = Uniaxial Compressive Strength; NA = Not available.
5.5 Stability analysis

The aim of this study was to apply different empirical and theoretical methods to assess the stability of a crown pillar at the site of investigation. At first, using the results from the joint survey and laboratory tests, three methods of rock mass classification (i.e., Bieniawski’s Geomechanics classification (RMR), the Q system and Laubscher’s Modified RMR system) and also Mathews stability graph method were used to assess the stability of 69NE2 crown pillar at the mine.

A comparison was made among the results obtained from the selected methods and it was found that none of them were solely adequate for the design of a crown pillar. Therefore, a combined empirical and theoretical method which was developed for the design of crown pillars (Section 3.5) was applied.

Numerical analysis was also carried out to assess the stability of the crown pillar from a stress analysis point of view. UDEC, a distinct element program, was used for the evaluation of various mining conditions and simulation of the open stoping operation in the region. To gain a better understanding of the failure mechanism, a series of parametric studies were also carried out using this program. To conclude, comparison of the condition predicted by selected methods and the conditions observed in the field was made.

5.5.1 Application of rock mass classification systems

A stable span was estimated for the crown pillar under study using the RMR system, the Q system, Laubscher's Modified RMR system and Mathews stability graph method. Although the stability graph method is not uniquely a rock mass classification system, it was included in this section due its use of a modified Q number (Q')as part of the analysis.

(a) Bieniawski’s Rock Mass Rating (RMR) system

The Geomechanics classification system is based on the following parameters:

1. Strength of intact rock material
2. Drill core quality (RQD)
3. Condition of joints
4. Spacing of joints
5. Ground water condition
Based on the data collected from the joint survey the RQD for the orebody was calculated as follows:

\[
x = 0.18 \text{ metres}
\]

Therefore, \( \lambda = 5.55 /\text{m} \)

\[
\text{RQD} = 100 e^{-0.1 \lambda} (0.1 \lambda + 1) = 89\%
\]

Where:

\[
x = \text{Average joint spacing}
\]

\[
\lambda = 1/x
\]

The method of determining the rock mass rating for the crown pillar is presented in Table 5.12. A RMR rating of 74, corresponding to rock class II which is considered as a good rock, was calculated. Relating the rating to the chart for estimation of safe span reveals that for a rock with a RMR equal to 74, a 12 metres span can remain unsupported for about 3 months.

**Table 5.12 Rock Mass Rating for the crown pillar at the site of investigation.**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial compressive strength</td>
<td>101 MPa</td>
<td>12</td>
</tr>
<tr>
<td>RQD</td>
<td>89%</td>
<td>17</td>
</tr>
<tr>
<td>Joint spacing</td>
<td>200 - 600 mm</td>
<td>10</td>
</tr>
<tr>
<td>Joint condition</td>
<td>slightly rough surfaces separation &lt;1 mm, hard joint wall rock</td>
<td>20</td>
</tr>
<tr>
<td>Ground water, inflow per 10 m tunnel length</td>
<td>dry, none</td>
<td>15</td>
</tr>
<tr>
<td>Total rating</td>
<td>-</td>
<td>74</td>
</tr>
</tbody>
</table>
(b) Q-system

As stated in Section 2.2.8, the Q system (Barton et al, 1974) which is based on rock mass quality relies on the following parameters:

1. RQD
2. Joint set number (Jn)
3. Joint alteration number (Ja)
4. Joint roughness number (Jr)
5. Joint water reduction factor (Jw)
6. Stress reduction factor (SRF)

\[ Q = \frac{RQD}{Jn} \cdot \frac{Jr}{Ja} \cdot \frac{Jw}{SRF} \]

Q values range from 0.001 to 1000. The following ratings can be attributed to the high grade ore at the site of investigation:

- **RQD** = 90
- **Jn rating** = 6 (two joint sets plus random joints)
- **Jr rating** = 3 (Rough and undulating)
- **Ja rating** = 0.75 (Tightly healed, hard and non-softening filling)
- **Jw rating** = 1 (Dry excavation)
- **SRF** = 2.5 (high stress condition)

\[ Q = \frac{90}{6} \cdot \frac{3}{1} \cdot \frac{1}{2.5} = 18 \]

\[ De = \frac{\text{Span}}{ESR} = \frac{12}{3} = 4 \] (For temporary mine opening, ESR = 3)

Where:
- **De** = Equivalent Dimension
- **ESR** = Excavation Support Ratio

With reference to Figure 2.4, the relationship between the maximum equivalent dimension, De, and the Q value, a 12 m span in a rock mass with a Q value equal to 18 will be stable without support.
(c) Laubscher's Modified RMR System

The modified RMR system was developed by Laubscher et al. (1976) to suit all mining situations. This system uses the same five parameters as Bieniawski's classification but has been modified in detail. The following parameters are used:

- RQD
- Intact Rock Strength (I.R.S.)
- Joint spacing
- Condition of joints and Ground water

The rock in the crown pillar can be characterised by the following ratings:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>RQD</td>
<td>90%</td>
</tr>
<tr>
<td>Intact Rock Strength</td>
<td>116 MPa</td>
</tr>
<tr>
<td>Joint Spacing:</td>
<td></td>
</tr>
<tr>
<td>Set 1 = 200 mm</td>
<td>rating = 13</td>
</tr>
<tr>
<td>Set 2 = 250 mm</td>
<td></td>
</tr>
<tr>
<td>Joint condition (including groundwater):</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>95% (wavy, unidirectional)</td>
</tr>
<tr>
<td>B</td>
<td>90% (rough)</td>
</tr>
<tr>
<td>C</td>
<td>100% (no alteration)</td>
</tr>
<tr>
<td>D</td>
<td>95% (non softening and coarse sheared material)</td>
</tr>
<tr>
<td>Rating</td>
<td>40 (A.B.C.D) = 32.5</td>
</tr>
<tr>
<td>Total rating</td>
<td>71.5</td>
</tr>
<tr>
<td>Total Adjustment</td>
<td>(Weathering) x (Strike and dip of orientation) x (Blasting) = (95% x 80% x 94%) = 71.5%</td>
</tr>
</tbody>
</table>

\[
\text{RMS} = \frac{(A-B)}{80} \times 0.8 \quad C
\]

Where:

\[
\begin{align*}
\text{RMS} &= \text{Rock Mass Strength} \\
A &= \text{Total rating} \\
B &= \text{Intact Rock Strength rating} \\
C &= \text{Intact Rock Strength} \\
\text{Total rating} &= 71.5 \\
\text{Subtract IRS rating, 71.5 - 12} &= 59.5
\end{align*}
\]
Determination of reduction factor, \( \frac{59.5}{80} \) = 0.74
Application of IRS in MPa, \( 116 \times 0.74 \) = 86.3
Correction to 80\%, \( 85.5 \times \frac{80}{100} \) = 69
Determination of adjustment percentage = 71.5 \%
Design Rock Mass Strength (DRMS) = 69 \times 71.5\% = 49.3 MPa

Considering the virgin stress conditions at CSA Mine as described in Section 5.1, using a modified tributary area theory (Figure 3.5 in Section 3.2.2) induced stresses in the crown pillar are obtained as follows:

- Maximum stress, \( \sigma_1 \) = 61 Mpa
- Minimum stress, \( \sigma_3 \) = 42.3 Mpa
- Maximum stress difference, \( \sigma_1 - \sigma_3 \) = 18.7 MPa

Using the support selection charts (Figure 2.6) the following results are obtained:

Chart (a) shows that with a maximum stress of 61 Mpa and a DRMS of 49.3 MPa, the rock falls in zone (III) (failure controlled) and the chart indicates that support is required to stabilise the crown pillar.

Chart (c) shows that with a stress difference of 18.7 MPa and a DRMS of 49.3 MPa, the rock falls in zone (II) (potentially unstable blocks) and the chart indicates that support is required to stabilise the key blocks in the crown pillar.

The worst condition is predicted when using chart (a). This chart indicates that spalling, rock falls, movement on joints and plastic deformation will occur in the crown pillar and rock reinforcement is required to stabilise the crown pillar (Laubscher, 1984). A support system can be chosen from chart (d) in Figure 2.6 which indicates that cable bolts and straps should be used for support of the crown pillar. It should be noted that the support chart (d) in this system generally looks at roadway roof support and it may not be adequate for the support of stopes and crown pillar. If the DRMS exceeds the mining environment stress which indicates no support is required, the support of key blocks which may be formed in the crown pillar should not be ignored.

(d) Application of the stability graph method

The stability graph method (Section 2.2.10) was also applied to assess the stability of the crown pillar under investigation. This method of designing open stopes has been used
successfully in many cases (Bawden et al, 1989). The data used for this analysis are as follows:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crown pillar span</td>
<td>12 m</td>
</tr>
<tr>
<td>Crown pillar length</td>
<td>22 m</td>
</tr>
<tr>
<td>Q system value</td>
<td>18</td>
</tr>
<tr>
<td>Initial horizontal stress</td>
<td>28.7 MPa</td>
</tr>
<tr>
<td>Induced maximum horizontal stress</td>
<td>61 MPa</td>
</tr>
</tbody>
</table>

The following values can be attributed to different factors for determination of the stability number:

\[
Q' = 45 \\
\text{Rock stress factor (A)} = 0.1 \\
\text{Rock defect orientation factor (B)} = 0.9 \\
\text{Design surface orientation factor (C)} = 1 \\
\text{Stability number (N)} = Q' \times A \times B \times C = 4.05 \\
\text{Shape factor (S)} = \frac{\text{Area}}{\text{Perimeter of exposed surface}} = 3.88
\]

The stability number (N) versus shape factor (S) is plotted on Figure 5.18 which shows that the crown pillar is located in the potentially unstable zone.

![Figure 5.18 Location of the 69NE2 crown pillar on the stability graph](image-url)
(e) Results

Crown pillar stability assessment was carried out using the RMR system, the Q system, Laubscher’s Modified RMR system and Mathews stability graph method. The results are presented in table 5.13. The discussion about the results obtained from these analysis will be made in Section 5.7.

Table 5.13 Stability estimation of a 12 m wide crown pillar in CSA Mine, Cobar.

<table>
<thead>
<tr>
<th>Rock mass classification system</th>
<th>Rating</th>
<th>Rock type</th>
<th>Stability</th>
<th>Type of support</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bieniawski’s RMR system</td>
<td>74</td>
<td>Class II, good rock</td>
<td>Stable for about 3 months</td>
<td>Not required</td>
</tr>
<tr>
<td>Q system</td>
<td>18</td>
<td>Good rock</td>
<td>Stable</td>
<td>Not required</td>
</tr>
<tr>
<td>MRMR system</td>
<td>DRMS = 51.4</td>
<td>good rock</td>
<td>Unstable</td>
<td>Rock bolts and cable bolts</td>
</tr>
<tr>
<td>Mathews Stability graph method</td>
<td>S = 3.88</td>
<td>-</td>
<td>Crown pillar is located in the potentially unstable zone</td>
<td>Width of pillar should be increased or stope height should be decreased</td>
</tr>
<tr>
<td></td>
<td>N = 4.05</td>
<td>-</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5.5.2 Assessment of crown pillar stability using the combined empirical and analytical method

In this section the combined empirical and analytical method, as proposed in section 3.6 is applied to assess the stability of the crown pillar at the site of investigation. In tabular mines like CSA Mine, the stope width normally equals the width of the orebody, therefore in this case the crown pillar is considered to have a 12 m span which is the average width of the orebody in the area under investigation. Based on this 12 m span, the following section will assess stability by comparing thickness, induced stress and rock mass strength. From this comparison a safety factor before and after the use of support can be determined. The safety factor is a direct indication of the degree of stability.
(a) Thickness estimation

The results of a computer run to determine the safe span against shear and buckling failure as a function of thickness for the conditions found at CSA are presented in Figure 5.19. This figure, based on voussoir beam theory which has been modified (Section 3.4) shows that a 12 m wide crown pillar with a thickness less than 1.8 m will buckle and one with a thickness greater than this may be stable. It should be noted that if the magnitude of the confining stress is low the crown may fail in shear, and if it is too high the crown will fail in compression.

![Figure 5.19 Determination of safe span for self supporting roof beam versus thickness.](image)

(b) Stress estimation

The virgin stress conditions at CSA Mine have been described by Maconochie et al (1981) This indicates that the east-west horizontal stress is the major principal stress and has a magnitude of 28.8 m at 550 m (Table 5.1). The dip angle of the orebody is about 80°. Using Equation 3.4 the maximum stress acting perpendicular to the orebody can be calculated:

$$\sigma_0 = \sigma_v \cos^2 \alpha + \sigma_h \sin^2 \alpha = 27.7 \text{ MPa}$$
Where:

\[ \sigma_0 = \text{Pre-mining stress normal to the orebody, MPa} \]
\[ \alpha = \text{Orebody inclination, degrees} \]

Considering different thicknesses for the crown pillar the mining induced stress was estimated using a modified tributary area method (Figure 3.5) and the results are presented in Table 5.14.

**Table 5.14** Stress induced in 12 m wide crown pillars of various thickness

<table>
<thead>
<tr>
<th>Crown pillar thickness, m</th>
<th>Stress, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>160</td>
</tr>
<tr>
<td>6</td>
<td>117</td>
</tr>
<tr>
<td>8</td>
<td>96.5</td>
</tr>
<tr>
<td>10</td>
<td>83.1</td>
</tr>
<tr>
<td>12</td>
<td>74.8</td>
</tr>
<tr>
<td>14</td>
<td>63.7</td>
</tr>
<tr>
<td>15</td>
<td>61</td>
</tr>
<tr>
<td>16</td>
<td>60.9</td>
</tr>
<tr>
<td>18</td>
<td>58</td>
</tr>
<tr>
<td>20</td>
<td>55</td>
</tr>
</tbody>
</table>

(c) Estimation of pillar strength

As explained before (Section 3.3.3) the formulas which have been used for determination of pillar strength underestimate the strength of pillars, therefore Equation 3.15 (Section 3.3.4) is used for determination of the strength of the rock mass in the crown pillar.

\[
\text{RMS} = \frac{\sigma_c (\text{RMR} - \sigma_c \text{ rating})}{80} \times 0.8 \\
\text{Eq. (3.15)}
\]

Where:

\[ \text{RMS} = \text{Rock Mass Strength, MPa} \]
\[ \sigma_c = 116 \text{ MPa (Intact rock strength)} \]
\[ \sigma_c \text{ rating} = 12 \text{ (Rating for Intact rock strength)} \]
\[ \text{RMR} = 71.5 \text{ (Rock Mass Rating)} \]
RMS \[= \frac{116 (71.5 - 12)}{80} \times 0.8 = 69 \text{ MPa}\]

The factor of safety is obtained by calculating the strength/stress ratio in the pillar. Figure 5.20 shows factor of safety versus thickness of an unsupported crown pillar at the site of investigation.

As described in Section 3.6, using Figures 3.5 and 3.25, the minimum thickness and bolt factor required to stabilise a crown pillar with a 12 m span is shown in Table 5.16. From this table it can be concluded that crown pillars with a thickness less than 8 m even with extensive support cannot be stabilised and pillars with thicknesses between 8 and 13 metres can be stabilised using bolting. Crown pillars with a thickness greater than 13 m are stable without support, although support of key blocks as always may be required.
Table 5.16 The minimum support required to stabilise crown pillars of various thicknesses

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>8 m</td>
<td>0.7</td>
<td>0.8</td>
<td>0.88</td>
<td>0.91</td>
<td>0.92</td>
<td>0.96</td>
<td>0.97</td>
<td>0.98</td>
</tr>
<tr>
<td>10 m</td>
<td>0.81</td>
<td>0.95</td>
<td>1.03</td>
<td>1.06</td>
<td>1.09</td>
<td>1.12</td>
<td>1.13</td>
<td>1.14</td>
</tr>
<tr>
<td>12 m</td>
<td>0.91</td>
<td>1.06</td>
<td>1.14</td>
<td>1.18</td>
<td>1.21</td>
<td>1.25</td>
<td>1.26</td>
<td>1.27</td>
</tr>
<tr>
<td>14 m</td>
<td>1.07</td>
<td>1.24</td>
<td>1.34</td>
<td>1.38</td>
<td>1.43</td>
<td>1.47</td>
<td>1.48</td>
<td>1.49</td>
</tr>
<tr>
<td>16 m</td>
<td>1.13</td>
<td>1.3</td>
<td>1.4</td>
<td>1.44</td>
<td>1.49</td>
<td>1.54</td>
<td>1.55</td>
<td>1.56</td>
</tr>
<tr>
<td>18 m</td>
<td>1.17</td>
<td>1.37</td>
<td>1.47</td>
<td>1.52</td>
<td>1.57</td>
<td>1.62</td>
<td>1.63</td>
<td>1.64</td>
</tr>
<tr>
<td>20 m</td>
<td>1.23</td>
<td>1.44</td>
<td>1.56</td>
<td>1.6</td>
<td>1.62</td>
<td>1.71</td>
<td>1.72</td>
<td>1.73</td>
</tr>
</tbody>
</table>

5.5.3 Numerical analysis

The distinct element program, UDEC, was used for the numerical analysis. Two particular problems were analysed as follows:

i. Stress analysis

ii. Factors (other than stress) which affect crown pillar stability.

The stress analysis was carried out to evaluate the overall stability and to determine the stress in the crown pillar at the site of investigation. The second analysis involving the factors which affect crown pillar stability included a sensitivity study to investigate the effect different parameters had on the maximum induced stress and the safety factor in the crown pillar. Also included was an analysis to determine the effect a secondary occurring but distinct shear zone within the orebody had on stability.

5.5.3.1 Crown pillar modelling

The ore body was modelled between the 740 and 575 sublevel of the mine. A primary model was developed based on the geological and mining parameters previously stated. Two sequences of mining were studied: Excavation of the upper 40 m stope, excavation of the lower stope (leaving the 15 metre crown pillar) and then the emplacement of backfill; the second sequence was excavation of the upper stope, fill and then excavate the lower stope.
Two major discontinuity sets were included in the model; one dipping at 80°, the other horizontal. UDEC can utilise three types of boundary condition; fixed boundary, stress boundary or special boundary elements. The fixed boundary tends to underestimate the true stress value in the critical region, whereas the stress boundary tends to overestimate. As the 'boundary distance' increases the fixed and stress boundaries converge to a limit around the value which the special boundary element type boundary predicts. Solutions using all three boundary types were considered in the analysis.

Individual blocks caused by the intersection of the discontinuities were fully deformable, could move relative to each other and also separate. UDEC automatically discretizes these individual blocks into triangular finite difference zones. Block interaction is analysed through the stresses and displacements at the common corner and edge contact points and the discontinuities themselves are treated as boundary conditions at block interfaces. A Mohr-Coulomb elastic-plastic failure criterion was used for blocks, whilst the Coulomb slip model was used for the joints.

The rock properties are shown in table 5.16 and using the following formulae are converted to the properties required as input data for modelling as shown in Tables 5.17.

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

Where:

- \(K\) = Bulk modulus, GPa
- \(G\) = Shear modulus, GPa
- \(E\) = Elastic modulus, GPa
- \(\nu\) = Poisson's ratio

**Table 5.16** Properties of rock types at the site of investigation

<table>
<thead>
<tr>
<th>Properties</th>
<th>Siltstone</th>
<th>ore</th>
<th>Black chloritic shear zone</th>
<th>Backfill material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus, GPa</td>
<td>58</td>
<td>85</td>
<td>42</td>
<td>0.15</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.35</td>
</tr>
<tr>
<td>Uniaxial compressive strength, Mpa</td>
<td>85</td>
<td>116</td>
<td>46</td>
<td>Not Required</td>
</tr>
</tbody>
</table>
Table 5.17 Data input for modelling the crown pillar

<table>
<thead>
<tr>
<th>Properties</th>
<th>Siltstone</th>
<th>ore</th>
<th>Black chloritic shear zone</th>
<th>Joint material</th>
<th>Fill material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk modulus, GPa</td>
<td>38.6</td>
<td>56.6</td>
<td>28</td>
<td>35.6</td>
<td>0.17</td>
</tr>
<tr>
<td>Shear modulus, GPa</td>
<td>23.2</td>
<td>30</td>
<td>16.8</td>
<td>25.6</td>
<td>.056</td>
</tr>
<tr>
<td>Cohesion, MPa</td>
<td>30</td>
<td>50</td>
<td>shear zone</td>
<td>0</td>
<td>.78</td>
</tr>
<tr>
<td>Friction angle, degree</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
<td>35</td>
</tr>
<tr>
<td>Density, kg/m³</td>
<td>2900</td>
<td>3500</td>
<td>2900</td>
<td>NR</td>
<td>2300</td>
</tr>
<tr>
<td>Condition</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

Note: Condition 3 = Elastic-Plastic condition.

The virgin stress condition at CSA as described by Maconochie et al (1981) is shown in Table 5.18.

Table 5.18 Virgin stress condition at CSA. Mine Cobar (after Maconochie et al 1981)

<table>
<thead>
<tr>
<th>Normal stress</th>
<th>Relative value</th>
<th>Value at 550 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>West</td>
<td>(\sigma_1 = 10 + 0.033Z)</td>
<td>28.7, MPa</td>
</tr>
<tr>
<td>North</td>
<td>(\sigma_2 = 2 + 0.030Z)</td>
<td>18.8, MPa</td>
</tr>
<tr>
<td>Vertical</td>
<td>(\sigma_v = 0 + 0.029Z)</td>
<td>16, MPa</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shear stress</th>
<th>Relative value</th>
<th>Value at 550 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical/West</td>
<td>(\tau_w = 0 + 0.012Z)</td>
<td>4.32, MPa</td>
</tr>
<tr>
<td>Vertical/North</td>
<td>(\tau_n = 0)</td>
<td>0, MPa</td>
</tr>
<tr>
<td>North/West</td>
<td>(\tau_{nw} = 0)</td>
<td>0, MPa</td>
</tr>
</tbody>
</table>

5.5.3.2 Fitting the model to the problem region

In order to obtain accurate results the geometry of the model must be sufficiently representative of the physical model. The first step is to ensure that the boundary of the model is far enough from the region of study so that the model results are not influenced by boundary effects. To determine a suitable distance for the model boundary, the problem was solved for various boundary distance from the stope walls. It can be seen from Figure 5.21 (a) that when the boundary is more than 10 stope widths away from the excavation
the maximum stress in the crown pillar approaches a constant value and with increasing distance of the boundary from the excavation the change of stress in the crown pillar is negligible. It can also be seen from Figure 5.21 (b) that movement of the hangingwall towards the footwall (convergence) also tends towards an equilibrium. These results indicate that for the purpose of this analysis the boundaries should be at a distance of at least 10 stope widths from the excavation on each side.

Figure 5.21 (a) Maximum horizontal stress versus boundary distance.

Figure 5.21 (b) Stope sidewall convergence versus boundary distance.
5.5.3.3 Stress analysis

The main point of this study was to determine the stress level expected in the crown pillar before and after excavation of the upper and lower stopes. In all cases the span and thickness of the crown were the same (12 m and 15m). The structural information was obtained from the field investigation and laboratory tests and was supplemented by data obtained from literature (Barton, 1978 and Worotniki, 1982). Using the model shown in Figure 5.22 the following two cases were analysed.

1) 12 m crown pillar span with 15m thickness (both stopes open)
2) case 1 + emplacement of fill material in the upper stope

![Figure 5.22 Basic model of crown pillar and stopes.](image)
Stable conditions in a UDEC analysis are shown when the algebraic sum of forces on a block equal zero or when the movement of a particular point approaches zero. Conversely unstable conditions are shown by unbalanced forces or when the velocity of a critical point is a non-zero constant. To reach a steady state the program uses timestepping, calculating the maximum unbalanced force for the whole model at each step and displaying it continuously on the screen. For each case which was analysed, after equilibrium, the maximum stress and displacement in the model were recorded and the history of the horizontal stress acting at and the velocity of a critical point in the centre of the crown pillar were monitored. Monitoring the velocity and movement of a critical point in the crown pillar shows that when the crown pillar is stable, movements increase during the mining stages and approach zero at the end of mining sequences.

In both cases, cable bolt effects were not included in the input data so unstable areas were allowed to deform. Results from the models were determined by looking at three histories:

<table>
<thead>
<tr>
<th>Location of point</th>
<th>Variable</th>
</tr>
</thead>
<tbody>
<tr>
<td>History 1</td>
<td>centre of the crown pillar</td>
</tr>
<tr>
<td>History 2</td>
<td>centre of the crown pillar</td>
</tr>
<tr>
<td>History 3</td>
<td>throughout the model</td>
</tr>
</tbody>
</table>

For case one (model with both stopes open) the following was found:

<table>
<thead>
<tr>
<th>Variable</th>
<th>Stability condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>History 1</td>
<td>Sxx (horizontal stresses)</td>
</tr>
<tr>
<td>History 2</td>
<td>Y velocity</td>
</tr>
<tr>
<td>History 3</td>
<td>Unbalanced force</td>
</tr>
</tbody>
</table>

For case two (model with fill material in the upper stope) the following was found:

<table>
<thead>
<tr>
<th>Variable</th>
<th>Stability condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>History 1</td>
<td>Sxx (horizontal stresses)</td>
</tr>
<tr>
<td>History 2</td>
<td>Y velocity</td>
</tr>
<tr>
<td>History 3</td>
<td>Unbalanced force</td>
</tr>
</tbody>
</table>

Figure 5.23 shows the history of the unbalanced forces acting on the model. The sum of these forces approaches zero at the end of the mining sequence, suggesting stability was
obtained. Figure 5.24 shows the principal stresses in the region after excavation of the upper stope. The main points to note are the horizontal destressing of the walls and the increase in horizontal stress in the back, and the region where the crown pillar will be formed. Figure 5.25 shows the change in horizontal stress in the crown pillar during the extraction of the top and bottom stopes. During extraction of the upper stope stress in the pillar (where the pillar will be formed to be precise) builds up until equilibrium is reached at the end of extraction. Stress builds up again as the lower stope is excavated and stabilises at a value of 74 MPa.

Figure 5.26 show the maximum and minimum principal stress in a 15 m thick crown pillar. This Figure shows that the maximum horizontal stress has increased to 74.6 MPa which is more than twice the initial stress and a compression arch has been formed in the model between two stopes which passes through the crown pillar. Analysis using a Mohr-Coulomb failure criteria indicates that the pillar is stable under these conditions. Figure 5.27 shows the principal stresses in the crown pillar at the end of the mining sequence and after backfilling the upper stope. As was expected, backfilling the upper stope at this point did not significantly alter the stress values in the crown pillar, hence the stability condition of the pillar was not affected.

In this case where the upper stope was backfilled before extracting the lower stope no significant effect on the maximum stress, decreasing from 74.9 MPa to 73.9 MPa, was found. This was also expected as the stiffness of the crown pillar was significantly greater than the backfill. Thus backfilling in this situation would serve only two purposes: Stabilising the hangingwall of the upper stope by limiting movement, and in turn helping maintain global stability after extraction of surrounding stopes. The data input file for stress analysis of the crown pillar has been shown in appendix 1.

- On the basis of the distinct element program a crown pillar of 12 m span and 15 m thickness at the site of investigation will be stable without support.

- Backfilling is not required in the case of 15 m crown pillars which remain stable as it reduces the stress by an only marginal amount. However, if failure of a particular pillar occurs, the backfill helps maintain global stability by accepting some of the redistributed stress and limiting sidewall movement.
Figure 5.23 History of unbalanced forces.

**UDEC (Version 1.83)**

**LEGEND**

8/10/1993 21:27
 cycle 1150
2.05E+04 <hist 1> 8.97E+06——

excavation of upper stope
excavation of lower stope

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Figure 5.26 Principal stress trajectories in the crown pillar at the end of mining sequence.

**UDEC (Version 1.82)**

**LEGEND**

12/07/1992 21:54  
cycle 1650

principal stresses  
minimum = -7.490E+07  
maximum = 1.026E-02

---

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Figure 5.27 Principal stresses in the crown pillar after backfilling the upper stope.

**UDEC (Version 1.82)**

**LEGEND**

3/09/1994 07:55
cycle 1150

principal stresses
minimum = -7.399E+07
maximum = -1.767E+05

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Figure 5.23 History of the horizontal stress in the lower crown pillar after failure of upper pillar.

**UDEC (Version 1.83)**

**LEGEND**

7/24/1993  15:00
cycle  2950
-7.80E+07 <hist 50> -5.61E+07

---

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University of Wollongong
Figure 5.29 Stress in the lower pillar after failing the upper pillar.

**UDEC (Version 1.83)**

**LEGEND**

7/24/1993 14:37

cycle 2950

principal stresses

- minimum = -8.461E+07
- maximum = 1.389E+00

boundary plot

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5.5.3.4 Sensitivity study of the factors affecting crown pillar stability

A better way of designing pillars needs a better understanding of failure mechanisms. Numerical modelling can be used to simulate the observed modes of failure and therefore can help in identifying the failure mechanism. All failure modes are not equally important from a mine planning point of view. It is not necessary to model instabilities which can be solved with systematic support (Sjoberg, 1992). In some cases the cause of instability is known to a large extent, such as a wedge which has been created by the intersection of joint planes.

It seems that a correct presentation of the mechanism and shape of the failure is more essential than calculating exact values of displacements or stresses. This is because in most cases the data input are not 100% accurate and therefore the exact values of stresses or displacement are not as useful as understanding the failure mode. Therefore, a series of parametric studies were carried out on models of 69NE2 crown pillar to investigate the effect various parameters have on stability in the crown pillar. For a particular series of models all but one of the following parameters; joint spacing, joint friction angle, orebody cohesion, elastic modulus and horizontal stress were kept constant so that the effect of each individual parameter could be monitored. The following values were taken as the base:

- Elastic modulus = 85 GPa
- Joint friction angle = 31°
- Orebody cohesion = 50 MPa
- Country rock cohesion = 34 MPa
- Orebody density = 3500 kg/m³
- Country rock density = 2900 kg/m³
- Insitu horizontal stress = 28.7 GPa
- Vertical stress = 16 GPa
- Joint normal stiffness = 50 GPa/m
- Joint shear stiffness = 5 GPa/m

Figure 5.22 illustrated the geometry of the stope and the crown pillar which was modelled. In all cases the dimensions of the stope and crown pillar were as follows:

- Stope height = 40 m
- Crown pillar span = 12 m

The maximum horizontal stress and factor of safety for the model with above values are 74 MPa and 1.3 respectively.
(a) Effect of vertical joint spacing

The results of this study are shown in Figure 5.31 (a) and (b) and Table 5.19. These results indicate that when joint spacing decreases stress in the crown pillar decreases and shear displacement and block movement in the crown pillar and stope walls also increase. Using a Mohr Coulomb failure criteria, the stress within the crown pillar was compared with its strength in order to determine the degree of stability of the pillar. The results of the comparison are shown in

Table 5.19 Summary of the analysis for different vertical joint spacing.

<table>
<thead>
<tr>
<th>Vertical joint spacing, m</th>
<th>Horizontal stress in centre of crown pillar, MPa</th>
<th>Max. shear contours in crown pillar, MPa</th>
<th>Factor of safety in the crown pillar</th>
<th>Monitoring the movement of a point in centre of crown pillar</th>
<th>Stope sidewalls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>72.1</td>
<td>60</td>
<td>1.3</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall Max = 42 mm</td>
</tr>
<tr>
<td>6</td>
<td>71</td>
<td>60</td>
<td>1.3</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall, Max = 42 mm</td>
</tr>
<tr>
<td>4</td>
<td>70</td>
<td>60</td>
<td>1.2</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall, Max = 42.2 mm</td>
</tr>
<tr>
<td>3</td>
<td>67</td>
<td>60</td>
<td>1.2</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall, max = 43 mm</td>
</tr>
<tr>
<td>2</td>
<td>66.5</td>
<td>60</td>
<td>1.1</td>
<td>increasing shear movement and plasticity in crown pillar</td>
<td>Hangingwall movement increased towards footwall, Max = 52 mm</td>
</tr>
<tr>
<td>1</td>
<td>66</td>
<td>60</td>
<td>1.0</td>
<td>increasing shear movement and plasticity, initiation of instability in the lower stope walls</td>
<td>Hangingwall movement towards footwall, Max=54.2 mm</td>
</tr>
</tbody>
</table>
Figure 5.31 (b) which indicates that the factor of safety against failure (degree of stability) decreases with decreasing joint spacing. It should be noted that even although the horizontal stress decreases, the strength of the pillar also decreases (with decreasing joint spacing), and as a result the safety factor decreases.

![Graph of horizontal stress versus vertical joint spacing.](image)

**Figure 5.31 (a)** Maximum horizontal stress versus vertical joint spacing.

![Graph of factor of safety versus vertical joint spacing.](image)

**Figure 5.31 (b)** Factor of safety versus vertical joint spacing.
(b) Effect of horizontal stress

To examine the effect of varying horizontal stress on the stability of the crown pillar, the insitu horizontal stress was calculated at different depths of the mine and the problem was solved for different values of horizontal stress. The results are presented in Figures 5.32 and 5.33 and Table 5.20.

Figure 5.32 Maximum horizontal stress versus insitu horizontal stress.

Figure 5.33 Factor of safety versus insitu horizontal stress.
The variation of insitu horizontal stress as shown in Figures 5.32 (a) and (b) corresponds to depths of 100 m to 700 m at the mine. The maximum horizontal stress in the crown pillar increases from 28.8 MPa to 88 MPa and the factor of safety decreases from 3 to 1. These results suggest that for a 10 % variation of insitu horizontal stress the change of maximum horizontal stress and factor of safety are 13.8 % and 4.4 % respectively. In other words for every 60 m increase of depth at the site, the maximum horizontal stress in the crown increases up by 5.9 MPa.

Table 5.20 Summary of the analysis for different insitu horizontal stresses

<table>
<thead>
<tr>
<th>Insitu horizontal stress, MPa</th>
<th>Max horizontal stress in the crown pillar, MPa</th>
<th>Horizontal stress in centre of crown pillar, MPa</th>
<th>Max shear stress MPa</th>
<th>Factor of safety in the crown pillar</th>
<th>Monitoring the movement of a point in centre of crown pillar</th>
<th>Stope walls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.3</td>
<td>28.8</td>
<td>25</td>
<td>20</td>
<td>3</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 36 mm</td>
</tr>
<tr>
<td>16.6</td>
<td>39</td>
<td>37</td>
<td>34</td>
<td>2.2</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 34 mm</td>
</tr>
<tr>
<td>19.9</td>
<td>49.1</td>
<td>47.7</td>
<td>40</td>
<td>1.7</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 35 mm</td>
</tr>
<tr>
<td>23.2</td>
<td>58.9</td>
<td>57.7</td>
<td>50</td>
<td>1.4</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max=36.5 mm</td>
</tr>
<tr>
<td>26.5</td>
<td>68.7</td>
<td>66.8</td>
<td>60</td>
<td>1.3</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 43 mm</td>
</tr>
<tr>
<td>29.8</td>
<td>78.8</td>
<td>76.5</td>
<td>65</td>
<td>1.1</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 43 mm</td>
</tr>
<tr>
<td>33.1</td>
<td>88</td>
<td>86.1</td>
<td>70</td>
<td>1</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 47 mm</td>
</tr>
</tbody>
</table>
(c) Effect of stope wall inclination

Results of the effect of variation of inclination of the stope wall on crown pillar stability are presented in Figures 5.34 (a) and (b) and Table 5.21.

![Graph showing maximum horizontal stress versus inclination of the stope wall.](image)

**Figure 5.34 (a)** Maximum horizontal stress versus inclination of the stope wall.

![Graph showing factor of safety versus inclination of the stope wall.](image)

**Figure 5.34 (b)** Factor of safety versus inclination of the stope wall.
Table 5.21 Summary of the analysis for different stope wall inclinations

<table>
<thead>
<tr>
<th>Stope walls inclination with respect to horizontal</th>
<th>Max horizontal stress in the crown pillar, MPa</th>
<th>Horizontal stress in centre of the crown pillar, MPa</th>
<th>Max shear stress, MPa</th>
<th>Factor of safety in the crown pillar</th>
<th>Monitoring the movement of a point in centre of crown pillar</th>
<th>Stope walls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi = 90^\circ$</td>
<td>75.9</td>
<td>74.3</td>
<td>60</td>
<td>1.2</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 40 mm</td>
</tr>
<tr>
<td>$\phi = 80^\circ$</td>
<td>74</td>
<td>72.1</td>
<td>60</td>
<td>1.3</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 42 mm</td>
</tr>
<tr>
<td>$\phi = 60^\circ$</td>
<td>60</td>
<td>60</td>
<td>50</td>
<td>1.4</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 49 mm</td>
</tr>
<tr>
<td>$\phi = 45^\circ$</td>
<td>50</td>
<td>50</td>
<td>40</td>
<td>1.8</td>
<td>crown pillar is stable but stope hangingwall is unstable</td>
<td>Hangingwall movement increased, unstable max=220 mm</td>
</tr>
</tbody>
</table>

These results show that when the dip of the stope walls increases, the horizontal stress in the crown pillar also increases. Conversely, a decrease in the dip of the stope wall can lead to potential instability in the hangingwall which may ultimately affect the stability of the crown pillar.

(d) Effect of elastic modulus of orebody

The effect of the change of the orebody elastic modulus 'E' on crown pillar stability is presented in Table 5.22 and Figures 5.35 (a) and (b). The results show that for a 10 % variation of this parameter the change of the maximum horizontal stress and factor of safety of the crown pillar in the model are 0.5 % and 0.15 % receptively.
Table 5.22 Summary of analysis for change of elastic modulus in orebody

<table>
<thead>
<tr>
<th>Elastic modulus GPa</th>
<th>Max horizontal stress in the crown pillar, MPa</th>
<th>Horizontal stress in centre of crown pillar, MPa</th>
<th>Max shear contours in crown pillar, MPa</th>
<th>Factor of safety in the crown pillar, MPa</th>
<th>Monitoring the movement of a point in centre of crown pillar</th>
<th>Stope sidewalls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>E = 10</td>
<td>57.2</td>
<td>56</td>
<td>40</td>
<td>1.6</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall (Max = 50 mm)</td>
</tr>
<tr>
<td>E = 20</td>
<td>65.1</td>
<td>64</td>
<td>50</td>
<td>1.4</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall (Max = 44 mm)</td>
</tr>
<tr>
<td>E = 40</td>
<td>70</td>
<td>69.6</td>
<td>60</td>
<td>1.35</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall (Max = 44 mm)</td>
</tr>
<tr>
<td>E = 60</td>
<td>72.9</td>
<td>71.8</td>
<td>60</td>
<td>1.35</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall (Max = 44 mm)</td>
</tr>
<tr>
<td>E = 70</td>
<td>73.4</td>
<td>71.8</td>
<td>60</td>
<td>1.3</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall (Max = 42 mm)</td>
</tr>
<tr>
<td>E = 85</td>
<td>74</td>
<td>72.1</td>
<td>60</td>
<td>1.3</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall (Max = 42 mm)</td>
</tr>
<tr>
<td>E = 100</td>
<td>74.4</td>
<td>72.5</td>
<td>60</td>
<td>1.2</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall (Max = 42 mm)</td>
</tr>
</tbody>
</table>
Figure 5.35 (a) Maximum horizontal stress versus elastic modulus.

Figure 5.35 (b) Factor of safety versus elastic modulus.
(e) Effect of orebody cohesion

The results of the analysis for variation of orebody cohesion are presented in Table 5.23 and Figures 5.36 (a) and (b). These results indicate that for cohesion values greater than 30 MPa the change of horizontal stress is not considerable while for lower values of cohesion the horizontal stress decreases rapidly and pillar failure will occur when cohesion values are less than 15 MPa. Figure 5.36 (b) shows that when the cohesion of rock material increases the safety factor also increases.

### Table 5.23 Summary of the analysis for different orebody cohesion

<table>
<thead>
<tr>
<th>Cohesion MPa</th>
<th>Max. horizontal stress in the crown pillar, MPa</th>
<th>Horizontal stress in centre of crown pillar, MPa</th>
<th>Max shear stress in crown pillar, MPa</th>
<th>Factor of safety in the crown pillar</th>
<th>Monitoring the movement of a point in centre of crown pillar</th>
<th>Stope walls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>C = 60</td>
<td>74</td>
<td>72.1</td>
<td>60</td>
<td>1.2</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 42 mm</td>
</tr>
<tr>
<td>C = 50</td>
<td>74</td>
<td>72.1</td>
<td>60</td>
<td>1.2</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 42 mm</td>
</tr>
<tr>
<td>C = 40</td>
<td>73</td>
<td>71</td>
<td>60</td>
<td>1.2</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 42.2 mm</td>
</tr>
<tr>
<td>C = 30</td>
<td>69.7</td>
<td>65.2</td>
<td>50</td>
<td>1.15</td>
<td>no significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall max = 42.8 mm</td>
</tr>
<tr>
<td>C = 20</td>
<td>51</td>
<td>49</td>
<td>36</td>
<td>1.1</td>
<td>tension and plasticity increased in crown pillar</td>
<td>Hangingwall movement towards footwall max = 44 mm</td>
</tr>
<tr>
<td>C = 15</td>
<td>42</td>
<td>35</td>
<td>30</td>
<td>1</td>
<td>crown pillar has failed</td>
<td>Hangingwall movement towards footwall max = 49 mm</td>
</tr>
</tbody>
</table>
The results show that when orebody cohesion is greater than 30 MPa for each 10% variation of this parameter the change of stress and factor of safety is 1.6% and 0.4% receptively. When cohesion is less than 30 MPa, the slope of the graphs of the change of maximum stress and factor of safety is sharper and in this area for 10% variation of orebody cohesion the change of maximum horizontal stress and factor of safety of the crown pillar in the model are 6.5% and 1.5%.

**Figure 5.36 (a)** Maximum horizontal stress versus orebody cohesion.

**Figure 5.36 (b)** Factor of safety versus orebody cohesion.
(f) Effect of the joint friction angle

The results of the analysis for various joint friction angles are presented in Figures 5.37 (a) and (b) and Table 5.23. These results indicate that the rock mass in the crown pillar has yielded for friction angles less than 20° and failure occurs when the friction angle is 15°. Figure 5.37 (b) shows that when the friction angle decreases the safety factor also decreases until failure occurs.

The results also show that when joint friction angle is greater than 25°, for each 10% variation the change of the maximum horizontal stress and factor of safety of the crown
pillar are 2% and 1.6%. When cohesion is less than 25\(^\circ\) the slope of the graphs of the change of maximum stress and factor of safety are sharper and in this area for each 10% variation of joint friction angle the change of maximum horizontal stress and factor of safety of the crown pillar are 11.2% and 3.5%. It should be noted that the value of joint friction angle as measured in the laboratory (Table 5.11) is 35\(^\circ\) and it is in the area in which the slope angle of the graphs are not very sharp.

### Table 5.24 Summary of the analysis for variation of joint friction angle

<table>
<thead>
<tr>
<th>Joint friction Angle</th>
<th>Max. horizontal stress in the crown pillar, MPa</th>
<th>Horizontal stress in centre of the crown pillar, MPa</th>
<th>Max shear stress in the crown pillar, MPa</th>
<th>Factor of safety in the crown pillar</th>
<th>Monitoring the movement of a point in centre of crown pillar</th>
<th>Stope walls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\phi = 50^\circ)</td>
<td>83</td>
<td>82</td>
<td>60</td>
<td>1.4</td>
<td>No significant movement, crown pillar is stable</td>
<td>Hangingwall and footwall movement toward each other, max = 38 mm.</td>
</tr>
<tr>
<td>(\phi = 45^\circ)</td>
<td>80</td>
<td>80</td>
<td>60</td>
<td>1.35</td>
<td>No significant movement, crown pillar is stable</td>
<td>Hangingwall and footwall movement toward each other, max = 40 mm.</td>
</tr>
<tr>
<td>(\phi = 40^\circ)</td>
<td>77.2</td>
<td>75</td>
<td>60</td>
<td>1.35</td>
<td>No significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall, max = 41 mm.</td>
</tr>
<tr>
<td>(\phi = 35^\circ)</td>
<td>74.6</td>
<td>72.1</td>
<td>60</td>
<td>1.3</td>
<td>No significant movement, crown pillar is stable</td>
<td>Hangingwall movement towards footwall, max = 42 mm.</td>
</tr>
<tr>
<td>(\phi = 30^\circ)</td>
<td>70</td>
<td>70</td>
<td>50</td>
<td>1.3</td>
<td>Plasticity increased in crown pillar, stable</td>
<td>Hangingwall movement towards footwall, max = 44 mm.</td>
</tr>
<tr>
<td>(\phi = 25^\circ)</td>
<td>69</td>
<td>67</td>
<td>50</td>
<td>1.2</td>
<td>Plasticity and movement increased in crown pillar</td>
<td>Hangingwall movement towards footwall, max = 46 mm.</td>
</tr>
<tr>
<td>(\phi = 20^\circ)</td>
<td>35.59</td>
<td>32</td>
<td>35</td>
<td>1.0</td>
<td>Significant movement, crown pillar is unstable</td>
<td>Increase of shear movement in hangingwall max = 50 mm.</td>
</tr>
<tr>
<td>(\phi = 15^\circ)</td>
<td>–</td>
<td>failed (22)</td>
<td>25</td>
<td>-</td>
<td>failed</td>
<td>-</td>
</tr>
</tbody>
</table>
(g) **Effect of crown pillar thickness on stability**

A series of models were developed to study the effect of the thickness of the crown pillar on stability. The basic model was similar to that used in section 5.6.1, i.e., the crown was taken as a 12 m horizontal beam separating two 40 m high stopes. The starting values of different parameters were also the same as previous model except the joint spacing which was changed in each run based on the thickness of the crown pillar to avoid the creation of blocks with a slenderness ratio greater than 5 which can affect the accuracy of the calculation in the program. The properties of the rock mass in and around the crown pillar were also taken from the basic model in section 5.6.1. During the study all of the above factors were kept constant whilst varying the thickness of the pillar. A summary of the results from this analysis are shown in Table 5.25 and Figures 5.38 (a) and (b). It should be noted that the average horizontal stress in the crown pillar is being compared in this case, not the maximum horizontal stress as in previous cases.

**Table 5.25** Summary of the analysis for different crown pillar thickness

<table>
<thead>
<tr>
<th>Crown pillar thickness, m</th>
<th>Average horizontal stress in the crown pillar, MPa</th>
<th>Safety factor in the crown pillar</th>
<th>Overall condition of the crown pillar</th>
<th>Stope wall movements</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>56</td>
<td>1.3</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, $\text{Max} = 52 \text{ mm}$</td>
</tr>
<tr>
<td>18</td>
<td>60</td>
<td>1.2</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, $\text{Max} = 53 \text{ mm}$</td>
</tr>
<tr>
<td>16</td>
<td>61</td>
<td>1.15</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, $\text{Max} = 54 \text{ mm}$</td>
</tr>
<tr>
<td>14</td>
<td>64</td>
<td>1.1</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, $\text{Max} = 61 \text{ mm}$</td>
</tr>
<tr>
<td>12</td>
<td>72</td>
<td>1</td>
<td>At yielding condition</td>
<td>Movement of stope walls towards each other, $\text{Max} = 61 \text{ mm}$</td>
</tr>
<tr>
<td>10</td>
<td>82</td>
<td>0.85</td>
<td>Failed</td>
<td>Instability increased in stope walls</td>
</tr>
</tbody>
</table>
The stability and stress condition in a 15 m thick crown pillar were discussed earlier (Section 5.6.3). Results of analysis of a 12 m thick crown pillar shows that when the thickness is 12 m the pillar is in a yielding condition. Figure 5.39 also shows the Mohr-Coulomb strength/stress contours inside the pillar under these conditions, and indicates that the safety factor in the crown is equal to 1. Analysis of crown pillars less than 12 m thickness shows failure.
Figure 5.39  Contours of Mohr-Coulomb inside a 12 m thick crown pillar.

**UDEC (Version 1.83)**

**LEGEND**

3/09/1994 12:25  
cycle 1150

M-C Strength/Stress Ratios  
contour interval= 1.000E-01  
number of contours/color= 4  
min= 1.000E+00  
1.000E+00  
1.300E+00  
1.500E+00

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The results show that the change of the crown pillar thickness has a significant influence on the stability of the crown pillar. Stable conditions were noticed when the thickness was greater than 14 m. Immediate failure occurred when the thickness was less than 12 m. It should be noted however that even with a thickness of 20 metres, the factor of safety is still only 1.3, suggesting that even this thickness of pillar would not be conducive to long term stability.

The discussion about the results obtained from the sensitivity study will be made in Section 5.7.

5.5.3.5 Effect of black chlorotic shear zone

Some of the failures at CSA Mine have been attributed to the occurrence of a black chlorotic shear zone (Section 5.2). Therefore, a third series of models were developed to analyse the effect of a shear zone running across the crown pillar between the hangingwall and the stope back or parallel to hangingwall and footwall. As with the primary model as described in section 5.6.3, the model was analysed with both stopes open and then with the upper stope backfilled. In the series four models were constructed to analyse the following cases:

(i) In case 1 the shear zone was assumed to occur at the contact between the hangingwall and the orebody.

(ii) In case 2 the shear zone was assumed to be semi-horizontal and intersect the back of the stope.

(iii) In case 3 the shear zone was assumed to run vertically through the crown pillar parallel to the hangingwall and footwall.

(iv) In case 4 the shear zone was assumed to be near horizontal extending between the hangingwall and footwall.

Table 5.26 shows the potential positions of the shear zone in the crown pillar. In the case of a flat dipping shear zone (case 2) the results indicate that after excavation of the lower stope some part of crown pillar will collapse. A schematic of the principal stresses and block velocity in the crown pillar after failure of blocks which are below the shear zone is shown in Figure 5.40. This indicates that although some part of the pillar has failed load is still being taken by the intact rock. The stress history of a point below the shear zone, Figure 5.41, indicates that after excavation of the upper stope the mass below the shear
zone tends to reach an equilibrium. After excavation of the lower stope the joints below the shear zone progressively fail and the stress tends to zero. In this case failure would be caused by a combination of shear and tension.

After failure of blocks below the shear zone, further iterations of the model show progressive failure of the crown pillar. This is illustrated in Figures 5.42 and 5.43 which show the stress history of a point and the block velocity above the shear zone. The stress history tending to zero indicates failure and a block velocity greater than zero indicates that equilibrium has not been reached (in this case also suggesting failure). A plot of the horizontal stress pattern in the crown pillar after 2000 iterations, illustrated in Figure 5.44, shows the gradual destressing of the area below the shear zone and the subsequent increase in stress concentration in the upper intact area of the pillar. As progressive failure of the pillar takes place, the stress concentration in the upper region increases holding the blocks together but eventually leading to overstressing. The purpose of any support system in this situation would be to limit shear along the joint and also to maintain the integrity of the pillar, thus helping the pillar to support itself and accept the redistributed stresses.

The mode of failure predicted by the model is consistent with the observed failure of a crown pillar in CSA Mine. In all other cases the crown pillar as shown in Table 5.26 was stable and there was no any significant movement.

Table 5.26 Results of stability analysis of a crown pillar with a shear zone.

<table>
<thead>
<tr>
<th>Location of shear zone</th>
<th>Graphical presentation of the shear zone inside the crown pillar</th>
<th>Stability assessment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1- In contact with hangingwall</td>
<td>![Graphical presentation]</td>
<td>Shear movement increased in the crown pillar but pillar is still stable.</td>
</tr>
<tr>
<td>Case 2- Semi-horizontal and intersect with hangingwall and back of the stope</td>
<td>![Graphical presentation]</td>
<td>Crown pillar is unstable and the part under the shear zone has collapsed</td>
</tr>
<tr>
<td>Case 3- Along the thickness and parallel with hangingwall and footwall</td>
<td>![Graphical presentation]</td>
<td>Crown pillar remains stable.</td>
</tr>
<tr>
<td>Case 4- Extended between hangingwall and footwall</td>
<td>![Graphical presentation]</td>
<td>Crown pillar is stable</td>
</tr>
</tbody>
</table>
Figure 5.40 Principal stresses and block velocity in the crown pillar with flat dipping shear zone after failure of blocks below the shear zone.

UDEC (Version 1.83)

LEGEND

7/19/1993 22:59
cycle 11000

block plot
principal stresses
minimum = -7.103E+07
maximum = 0.000E+00

velocity vectors
maximum = 1.230E+00

Case 2, flat dipping
shear zone in the crown pillar

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Figure 5.41 Stress history of a point below the shear zone, case 2, flat dipping shear zone in the crown pillar.

**UDEC (Version 1.83)**

**LEGEND**

7/19/1993 14:03
cycle 1400
-3.17E+07 <hist 2>-2.66E+07

Case 2, flat dipping shear zone in the crown pillar

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Figures 5.42 Stress history of a point above the shear zone.

**UDEC (Version 1.83)**

**LEGEND**

7/19/1993 13:57
cycle 1900
-4.77E+07 <hist 11> 0.00E+00

Case 2, flat dipping shear zone in the crown pillar

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Figures 5.43 Principal stresses and block velocity above the shear zone.

UDEC (Version 1.83)

LEGEND

7/19/1993 13:53

cycle 1900

block plot
principal stresses
- minimum = -8.313E+07
- maximum = 3.483E-03

velocity vectors
- maximum = 7.363E-02

Case 2, flat dipping
shear zone in the crown pillar

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Figure 5.44 Destressing of the area below the shear zone and the increase in the stress in the upper intact area of the pillar.

**UDEC (Version 1.83)**

**LEGEND**

- 7/24/1993 20:29
- cycle 2000
- xx-stress contours
  - contour interval = 1.000E+07
  - min = -6.000E+07 max = 0.000E+00
- block plot

Case 2, flat dipping shear zone in the crown pillar

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5.6 Discussion

Three methods of rock mass classification, i.e., RMR, Q, and Modified RMR system in addition to the stability graph method were used to assess crown pillar stability. Bieniawski’s RMR system predicts that the crown pillar under investigation would have been stable for 3 months and no support would be required. In reality, the time to extract and fill a stope is longer than this period; generally between 9 and 12 months. Therefore, the RMR system suggests that the crown pillar should be supported to enable it to remain stable during the life of the stope. This tends to agree with the reality of the situation at the site, as even though support was installed there was evidence of pillar degradation with time. This would certainly have been more severe if no support was installed.

Based on the results obtained from the Q system, the crown pillars under investigation should be stable and support would not be required. However, because the results obtained from this analysis were very close to the border line which separates the stable and unstable zones, support would still be recommended to increase the factor of safety. The problem with the Q system in this case is that it takes no account of the time-dependent properties of the pillar. As such, it is left to the engineer to decide whether the close proximity to the unstable zone constitutes a problem and decide if it would be prudent to use some form of support.

Contrary to the Q system and the RMR system, the Modified RMR-system indicates that the crown pillar, due to large differences between the major and minor principal stresses, may fail if not supported. Supporting the rock with cable bolts and straps improves the DRMS and increases the confining pressure in the rock mass. As a result, the stress differences will decrease and the safety factor against failure will increase. It would appear that this system provides a more cautious analysis of the rock mass’s ability to support itself. Also, this system is awkward to use and it is doubtful if many site engineers would take the trouble to use it.

Results of Mathews stability graph method show that the crown pillar is located in the potentially unstable zone. Although no support system was recommended directly by this system, support requirements can be determined using a method suggested by Potvin (1988) cited Bawden et al (1989). Potvin provides a table of bolting factors (metres of cable/m²) which will "increase" the Q value of the rock mass thus providing an apparent increase in the quality of the rock mass and taking it into the stable zone. As proposed by Potvin’s method using a bolt factor of about 2 metres of cable/m² relocates the crown pillar under study to a stable position in the stability graph. It should be pointed out that in
this mine the support was installed in fans from a driveage, whereas Potvin's chart is designed for a more systematic pattern.

The results obtained from the analysis by the combined empirical and theoretical method shows that a 12 m wide crown pillar, 15 m thick is stable without support. However in this case the safety factor is only 1.1 which is relatively low and may cause problems for long term stability. Therefore support is required to improve the factor of safety in the pillar so that it can remain stable for a longer period of time. Table 5.16 shows the change of the factor of safety in a 12 m wide crown pillar by using cable-bolting. Using a bolt factor of 3.5 metres of cable bolt/m² is comparable with reality and gives a safety factor of about 1.4. Generally the crown pillar at the site are supported with 6 to 8 cable bolts of between 6 m to 10 m in length. This method shows that bolting improves the factor of safety in pillars, and even pillars between 8 and 13 m thick which would fail without support, can be stabilised by bolting. This also suggests that instead of increasing the thickness, the pillar can be bolted to obtain the same level of safety. For example when the thickness increases from 15 m to 20 metres (Table 5.16), the factor of safety of an unsupported pillar changes from 1.10 to 1.23 while supporting a 15 m thick crown pillar with a bolt factor of about 1 metre of cable bolt/m², increases the factor of safety to the same value.

The advantage of the combined empirical and theoretical method over other design methods is that it can predict a safe span as well as the optimum thickness for crown pillars. The comparison of the results of this method with the results of numerical analysis show that both methods predict that crown pillars greater than 14 m thick are stable and unsupported pillars less than 12 m thick will fail under this conditions. In Figure 5.45 the stress in the crown pillar calculated by numerical analysis for different thicknesses has been compared with that determined from the combined method. The safety factors obtained from both methods are shown in Figure 5.46 and it can be seen that the results in both cases are comparable.
Variation of all parameters considered in the sensitivity study can affect the maximum horizontal stress and the stability of the crown pillar, however, the levels of their effect are different. Comparison of the results show that initial horizontal stress is the most important parameter and any error in measurement of this parameter will have significant effect on the analysis. The depth of the orebody is also an important factor (directly
affects horizontal stress) and this study indicates that at the site, for every 60 m increase of depth, the maximum horizontal stress in the crown pillar increase by 5.9 MPa.

The results of the sensitivity study for the crown pillar at the site of investigation are shown in Table 5.27. This table shows that any error in measurement of initial horizontal stress of ±10% will have significant effect on stability analysis while the same percentage error in the measurement of cohesion or elastic modulus will not have significant effect on the results of the analysis.

Table 5.27 Results of sensitivity study for the crown pillar at the site of investigation

<table>
<thead>
<tr>
<th>Variation of parameter (±10%)</th>
<th>Change of factor of safety (%)</th>
<th>Variation of maximum horizontal stress (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Insitu horizontal stress</td>
<td>4.4</td>
<td>13.8</td>
</tr>
<tr>
<td>Joint friction angle</td>
<td>1.6</td>
<td>2</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>0.15</td>
<td>0.6</td>
</tr>
<tr>
<td>Orebody cohesion</td>
<td>0.4</td>
<td>1.6</td>
</tr>
</tbody>
</table>

However for reduced values of joint friction angle and cohesion as shown in Table 5.28 the effects of variation of these parameters on stability are considerable and any error in measurement would have significant effect on the analysis.

Table 5.28 Results of sensitivity study for the lower range of values selected parameters

<table>
<thead>
<tr>
<th>Variation of parameter (±10%)</th>
<th>Change of factor of safety (%)</th>
<th>Variation of maximum horizontal stress (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint friction angle &lt; 25°</td>
<td>3.5</td>
<td>11.3</td>
</tr>
<tr>
<td>Orebody cohesion &lt; 30 MPa</td>
<td>1.5</td>
<td>6.5</td>
</tr>
</tbody>
</table>
5.7 Conclusion

The followings are the most important conclusions of this study:

- All rock mass classification systems analysed in this study provided a fair indication of the quality of the rock mass constituting the pillar, although some methods were more conservative than others. The RMR - System indicates that support would be required if the crown pillar was to remain stable during the life of the stope. The Q - System suggests that no support would be required per se, however, the judgment of the engineer would be required due to the close proximity of the rock mass to the unstable zone. The MRMR - System seemed to be a bit conservative when compared with reality. It is also a very 'extravagant' system which a lot of site engineers would not take the time to decipher. Mathews stability graph method has the advantage of relating the span of the back to the strike length. When coupled with Potvin's bolting factors it is a fairly simple and practical method to use. The results obtained from a combination of these methods was in fair agreement with reality.

- All the rock mass classification systems used in this study have the disadvantage, for crown pillar design, that they are only really relevant to the design of the stope back. They provide no information on the required pillar thickness, and it is therefore necessary to use one of the above systems in conjunction with some standard form of pillar design. The other limitation of these systems is that they do not take account of the formation of potentially unstable wedges. This requires the use of some form of analysis which can identify isolated wedges. At CSA Mine the formation of isolated wedges was a major problem.

- Results of numerical analysis using the UDEC program indicated that crown pillars greater than 14 m thick are generally stable and crown pillars between 12 m and 14 m thick are in the yield zone and should be supported to maintain stability.

- The results from the sensitivity study indicated that the stability condition of the crown pillar was very sensitive to depth and the initial horizontal stress. Elastic modulus and cohesion have little influence on stability, but a reduced friction angle has significant influence on stability. Increasing the thickness makes the crown pillar more stable, not so much by increasing the strength of the pillar but by reducing the maximum stress within. Closely spaced joint sets form small blocks and as a consequence block movement increases in the crown pillar and safety factor decreases. This study also indicates that when the inclination of the stope walls increase the stress in the crown
pillar also increases, however, a decrease in the inclination of the stope walls leads to potential instability in the hangingwall which can later affect the stability of the crown pillar.

- This study also emphasises the importance of the accurate measurement of insitu horizontal stress and it indicates that any error in measurement of this parameter has significant effect on the analysis. Joint friction angle is also relatively an impotent factor and should be measured carefully in the field or laboratory while small percentage of error in the measurement of cohesion or elastic modulus will not have significant effect on the results of the analysis. It should be noted that the lower values of cohesion and elastic modulus have significant effect on stability and in this case the accurate measurements of theses parameters is important.

- The shear zone running from the hangingwall to the bottom face of the crown pillar led to progressive failure of the area below the shear zone, eventually causing overstressing of the remaining intact pillar. In general installation of cable bolts can improve the situation by increasing the resistance to shear and tensile failure and helping to maintain the integrity of the pillar. Support of the crown pillar also provides a confinement stress in the minimum principal stress direction (vertical), therefore, the stress difference between the maximum and minimum principal stresses is less and although there is a very high stress inside the crown pillar, the pillar will be stronger due to the action of the cable bolts. Knowledge of the actual action of the support system and whether it will be reliable enough to ensure pillar stability is dependent on monitoring the crown pillar in the field after installation of support.

- If it was decided to dispense with the use of backfill when utilising a standard 15 m pillar, it would be imperative that any support system was adequately installed and would help maintain the integrity of the crown pillar. This would be critical in crown pillars which had weak shear zones running across them.

- Field observations at CSA mine show that most crown pillars of 15 m thickness are stable with the help of cable bolts. The results predicted by numerical analysis indicated that a 15 m thick crown pillar is stable without support. However, a support system in this situation would be required to resist shear and tensile forces in the joints, thus locking in key blocks, thereby helping the pillar to support itself and accept the redistributed stresses.
The combined empirical and theoretical method developed for estimation of crown pillar geometry and assessment of stability, when applied to CSA Mine, predicted that a 15 m thick crown pillar would have a factor of safety equal to 1.1 without support. Because the safety factor is relatively low it may cause stability problem in the long term. Therefore support would be required to increase the factor of safety and improve the crown pillar potential for longer term stability.

When predicting stress level in the orebody, the combined empirical and theoretical method compared favourably with numerical analysis. The method was able to provide a good estimations of the stress level for a wide range of crown pillar thicknesses. This method is particularly useful for the estimation of stress and the thickness of crown pillars at the feasibility stage or as an initial estimate during normal operations. It is obvious that for actual design numerical methods should be used to verify results obtained from the combined method, and also to predict stress level in more complex situations.
HOLE DEPTH WHERE NO MINED AREAS EXIST ABOVE STOPES

TO BE DRILLED IN ALL CASES

HOLE DEPTH WHERE BRIDGE PILLAR LEFT

NOTE:
- hole diameter 64 to 70 mm
- rings are aligned with
- blast hole rings
- depth of hole no.

Figure 6.4 (a) Generalised cable bolt pattern for the crown pillar at NBHC
Figure 6. 4 (b) Typical support of backs and hangingwall by cable bolts. (after Hunt and Askew, 1977)

6.3 Field study

As the first step of this investigation, a joint survey was carried out at the site of investigation to determine the structural characteristics of the crown pillar. Six scanlines were used, three were 10 m in length parallel to the floor, and three were vertical and perpendicular to the three others. The data were collected by analysing the discontinuities intersecting the scanline. The results of joint survey were used to determine the potential for individual wedge failure. Figure 6.5 is a plan view of the area where the joint survey took place.
The discontinuity orientations were plotted on a Schmidt equal area lower hemisphere stereonet. The DIPS program was used to obtain results from the joint survey data. This program which assesses the structural stability of blocks intersected by joint sets was developed by Hoek and Diederichs (1989). Figure 6.6 is a scatter plot of pole concentrations at the site of investigation. Figures 6.7 and 6.8 show the contour diagram and plane of the major joint sets for the this area. A near vertical and a flat dipping joint set are the most important structural features which were distinguished in this investigation. The dip of the horizontal joint set is about 15°, bearing 023°, and the vertical joint set has an average dip of 70° bearing 137°. A third, random joint set is evident and dips at around 78°, bearing 195°. A third, random joint set is evident and dips at around 78°, bearing 195°.
Figure 6.6 Scatter plot of pole concentrations at the site of investigation.

Figure 6.7 Contour diagram of pole concentrations at the site of investigation.
Potentially unstable wedges are formed by the intersection of the three joint sets, Figure 6.8. Using Lucas (1980) method for predicting wedge failure it can be determined that this wedge can fall vertically under gravity (the azimuths of the three intersection vectors do not lie within 180° of arc).

Figure 6.8 Plane of major joint sets at the site of investigation.

6.4 Rock properties and laboratory rock testing

Information on the mechanical properties of the rocks in this mine were obtained from a series of tests which were carried out in the Rock Mechanics Laboratory, University of Wollongong.

(a) Uniaxial Compressive Strength

The results of the tests on high grade ore and country rock (garnet quartzite) are presented in Tables 6.3 and 6.4
## Table 6.3 Data obtained from uniaxial tests on country rock

<table>
<thead>
<tr>
<th>Test No.</th>
<th>U.C.S., MPa</th>
<th>Poisson’s ratio</th>
<th>Elastic modulus, GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>78</td>
<td>0.11</td>
<td>44</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
<td>0.11</td>
<td>60</td>
</tr>
<tr>
<td>3</td>
<td>111</td>
<td>0.12</td>
<td>98</td>
</tr>
<tr>
<td>4</td>
<td>112</td>
<td>0.30</td>
<td>65</td>
</tr>
<tr>
<td>5</td>
<td>120</td>
<td>0.20</td>
<td>78</td>
</tr>
<tr>
<td>6</td>
<td>120</td>
<td>0.31</td>
<td>94</td>
</tr>
<tr>
<td>7</td>
<td>116</td>
<td>0.16</td>
<td>59</td>
</tr>
<tr>
<td>8</td>
<td>102</td>
<td>0.26</td>
<td>102</td>
</tr>
<tr>
<td>9</td>
<td>80</td>
<td>0.23</td>
<td>113</td>
</tr>
<tr>
<td>10</td>
<td>152</td>
<td>0.20</td>
<td>65</td>
</tr>
<tr>
<td>11</td>
<td>168</td>
<td>0.10</td>
<td>61</td>
</tr>
<tr>
<td>12</td>
<td>170</td>
<td>0.30</td>
<td>103</td>
</tr>
<tr>
<td>13</td>
<td>60</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>94</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>15</td>
<td>149</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>16</td>
<td>159</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>17</td>
<td>71</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>18</td>
<td>193</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>19</td>
<td>85</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Average</td>
<td>117</td>
<td>0.2</td>
<td>78.5</td>
</tr>
</tbody>
</table>
Table 6.4 Data obtained from uniaxial test on ore

<table>
<thead>
<tr>
<th>Test No.</th>
<th>U.C.S., MPa</th>
<th>Poisson’s ratio</th>
<th>Elastic modulus, GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>68</td>
<td>0.35</td>
<td>43</td>
</tr>
<tr>
<td>2</td>
<td>114</td>
<td>0.24</td>
<td>41</td>
</tr>
<tr>
<td>3</td>
<td>81</td>
<td>0.21</td>
<td>42</td>
</tr>
<tr>
<td>4</td>
<td>96</td>
<td>0.17</td>
<td>88</td>
</tr>
<tr>
<td>5</td>
<td>122</td>
<td>0.24</td>
<td>93</td>
</tr>
<tr>
<td>6</td>
<td>75</td>
<td>0.28</td>
<td>74</td>
</tr>
<tr>
<td>7</td>
<td>90</td>
<td>0.40</td>
<td>71</td>
</tr>
<tr>
<td>8</td>
<td>64</td>
<td>0.20</td>
<td>44</td>
</tr>
<tr>
<td>9</td>
<td>97</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>88</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>78</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>129</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>77</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>102</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>77</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>103</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>132</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>113</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>107</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>97</td>
<td>0.26</td>
<td>62</td>
</tr>
</tbody>
</table>
(b) Point Load test

The point load test was used to estimate indirectly the uniaxial compressive strength of the rocks. The results of these tests for ore and country rock are presented in Tables 6.5 (a) and (b).

Table 6.5 (a) Results of point load tests on ore

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Diametric test</th>
<th>Axial test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \sigma_C ), MPa</td>
<td>( \sigma_C ), MPa</td>
</tr>
<tr>
<td>1</td>
<td>65</td>
<td>65</td>
</tr>
<tr>
<td>2</td>
<td>41</td>
<td>90</td>
</tr>
<tr>
<td>3</td>
<td>46</td>
<td>86</td>
</tr>
<tr>
<td>4</td>
<td>172</td>
<td>82</td>
</tr>
<tr>
<td>5</td>
<td>49</td>
<td>102</td>
</tr>
<tr>
<td>6</td>
<td>99</td>
<td>107</td>
</tr>
<tr>
<td>7</td>
<td>41</td>
<td>153</td>
</tr>
<tr>
<td>8</td>
<td>107</td>
<td>90</td>
</tr>
<tr>
<td>9</td>
<td>49</td>
<td>74</td>
</tr>
<tr>
<td>10</td>
<td>127</td>
<td>131</td>
</tr>
<tr>
<td>11</td>
<td>-</td>
<td>82</td>
</tr>
<tr>
<td>Average</td>
<td>80</td>
<td>96.5</td>
</tr>
</tbody>
</table>

Table 6.5.(b) Results of point load test on country rock

<table>
<thead>
<tr>
<th>Name of test</th>
<th>Diametric test</th>
<th>Axial test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test No.</td>
<td>( \sigma_C ), MPa</td>
<td>( \sigma_C ), MPa</td>
</tr>
<tr>
<td>1</td>
<td>115</td>
<td>107</td>
</tr>
<tr>
<td>2</td>
<td>123</td>
<td>169</td>
</tr>
<tr>
<td>3</td>
<td>134</td>
<td>95</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>130</td>
</tr>
<tr>
<td>5</td>
<td>-</td>
<td>123</td>
</tr>
<tr>
<td>6</td>
<td>-</td>
<td>74</td>
</tr>
<tr>
<td>7</td>
<td>-</td>
<td>74</td>
</tr>
<tr>
<td>Average</td>
<td>124</td>
<td>110</td>
</tr>
</tbody>
</table>
(c) Triaxial Test

The results of this test for different rocks at the site of investigation are presented in Tables 6.6 and 6.7 and Figures 6.9 to 6.12.

Table 6.6  Data obtained from triaxial tests on country rock

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Confining stress, MPa</th>
<th>Normal stress, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>150</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>165</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>185</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>178</td>
</tr>
<tr>
<td>5</td>
<td>11</td>
<td>186</td>
</tr>
<tr>
<td>6</td>
<td>14</td>
<td>222</td>
</tr>
</tbody>
</table>

Figure 6.9  Strength envelope of country rock.
Figure 6.9 Construction of Mohr's envelope for country rock.
Chapter 6. Investigation into the stability of crown pillars in a lead-zinc mine - a case study

Table 6.7 Data obtained from triaxial tests on ore

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Confining stress ($\sigma_3$) MPa</th>
<th>Normal stress ($\sigma_1$) MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>111</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>163</td>
</tr>
<tr>
<td>3</td>
<td>15</td>
<td>191</td>
</tr>
</tbody>
</table>

![Graph](image)

Figure 6.11 Strength envelope of ore.

(d) Direct Shear Test

This test is used to investigate the shear strength of intact soft rock, planes of weakness and the frictional properties of discontinuities. The results of these tests are presented in Table 6.8 and Figures 6.13 and 6.14.

Table 6.8 Results of field shear box tests on ore

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Normal load, MN</th>
<th>$\sigma_n$, MPa</th>
<th>Shear stress, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>2.03</td>
<td>5.2</td>
</tr>
<tr>
<td>2</td>
<td>9</td>
<td>3.8</td>
<td>8.4</td>
</tr>
<tr>
<td>3</td>
<td>13</td>
<td>5.67</td>
<td>11.35</td>
</tr>
</tbody>
</table>
Figure 6.13 Maximum shear stress versus normal stress for ore.

Figure 6.14 Results of shear displacement versus shear stress, ore.
6.4.1 General results

The strength and deformation properties of rock obtained from the laboratory tests are presented in Table 6.9. The results indicate that high grade ore is moderately strong and the garnet quartzite is a strong rock. As was expected the orebody shows lower strength than the country rock due to its coarse grain and friable nature. Determination of the elastic properties shows that the country rock has an elastic modulus averaging 78.5 GPa and a Poisson's ratio of 0.2. The elastic modulus of the orebody is 62 GPa and Poisson's ratio 0.26. It must be considered that the above values are valid for the lower lead lode only because the sample were taken from that part of orebody.

Table 6.9 Mechanical properties of rocks in lower lead lode

<table>
<thead>
<tr>
<th>Rock properties</th>
<th>Type of test</th>
<th>Garnet quartzite</th>
<th>High grade ore</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS, MPa</td>
<td>Uniaxial test</td>
<td>117</td>
<td>97</td>
</tr>
<tr>
<td>UCS, MPa</td>
<td>Point load test</td>
<td>152</td>
<td>89</td>
</tr>
<tr>
<td>Elastic modulus, GPa</td>
<td>Uniaxial test</td>
<td>78.5</td>
<td>62</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>Uniaxial test</td>
<td>0.2</td>
<td>0.26</td>
</tr>
<tr>
<td>Internal friction angle, Degrees</td>
<td>Triaxial test, derived from strength envelope</td>
<td>42</td>
<td>47</td>
</tr>
<tr>
<td>Apparent cohesion (C), MPa</td>
<td>Triaxial test, Mohr's envelope</td>
<td>30</td>
<td>18</td>
</tr>
<tr>
<td>Apparent cohesion (C), MPa</td>
<td>Triaxial test, derived from strength envelope</td>
<td>32</td>
<td>16</td>
</tr>
<tr>
<td>Joint friction angle, Degrees</td>
<td>Field shear box</td>
<td>35</td>
<td>31</td>
</tr>
<tr>
<td>Density, Kg/m³</td>
<td>-</td>
<td>2900</td>
<td>3500</td>
</tr>
</tbody>
</table>
6.5 Stability analysis

As with the previous case study the aim was to apply different empirical and theoretical methods to assess the stability of a crown pillar at the site of investigation. The three methods of rock mass classification described previously were again used as well as Mathews stability graph method and distinct element program UDEC. The combined empirical and theoretical method which (Section 3.5) was also applied to the site and all the results compared the field observations.

6.5.1 Application of rock mass classification systems

Different rock mass classification systems were applied to assess the stability of crown pillar at the site of investigation The methods used were; the Bieniawski’s Geomechanics Classification, the Q system, Laubscher’s Modified RMR system and Mathews stability graph method.

(a) Bieniawski’s RMR system

Based on the data collected from the joint survey the RQD for orebody was calculated using Equation 4.3:

\[ x = 0.3, \text{ m} \]

Therefore, \( \lambda = \frac{3.25}{\text{m}} \)

\[ \text{RQD} = 100 e^{-0.1\lambda (0.1\lambda + 1)} = 95\% \]

Where:

\[ x = \text{Average joint spacing} \]
\[ \lambda = \frac{1}{x} \]

The RMR was determined from the following:

1) UCS = 97, rating = 7
2) RQD = 95 %, rating = 20
3) Joint spacing = 300 mm, rating = 10
4) Joint condition = Slightly rough surfaces, separation <1 mm, rating = 20
5) Ground water = Damp, rating = 10
6) Total RMR = 67

A RMR rating of 67 corresponding to rock class II (considered as a good rock) was determined. Relating the rating to safe span and stand up time reveals that for rock with a RMR equal to 67, a 16 m span can remain unsupported for about 2 months.

(b) Q system

Calculations pertinent to the Q-system are as follows:

\[
\begin{align*}
\text{RQD} &= 95 \\
\text{Jn rating} &= 6, \text{ (two joint sets plus random joints)} \\
\text{Jr rating} &= 3, \text{ (rough or irregular, undulating)} \\
\text{Ja rating} &= 1, \text{ (unaltered joint walls, surface staining only)} \\
\text{Jw rating} &= 1, \text{ (Dry excavation or minor flow)} \\
\text{SRF} &= 2.5, \text{ (high stress condition)} \\
\text{Q} &= \left(\frac{95}{6}\right) \cdot \left(\frac{3}{1}\right) \cdot \left(\frac{1}{2.5}\right) = 23.7 \\
\text{De} &= \text{Span/ESR} = \frac{16}{3} = 5.3 \text{ (For temporary mine openings; ESR = 3)}
\end{align*}
\]

Where:

\[
\begin{align*}
\text{De} &= \text{Equivalent Dimension} \\
\text{ESR} &= \text{Excavation Support Ratio}
\end{align*}
\]

With reference to Figure 2.4 which shows, the relationship between the maximum equivalent dimension, De, and the Q value, a 16 m span in a rock mass with a Q value equal to 23.7 will be stable without support.

(c) Application of Laubscher’s Modified RMR system

The following ratings can be attributed to the high grade ore at the site of investigation:

<table>
<thead>
<tr>
<th>Property</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>RQD</td>
<td>95%</td>
</tr>
<tr>
<td>Intact Rock Strength</td>
<td>97 MPa</td>
</tr>
<tr>
<td>Joint Spacing (two joint sets)</td>
<td>200 - 600 mm</td>
</tr>
</tbody>
</table>
Joint condition and groundwater:

A = 95% (wavy, unidirectional)
B = 90% (rough)
C = 100% (No alteration)
D = 90% (non-softening and sheared material, clay and talc free)

Rating = 40 . (A.B.C.D) = 31
Total rating = 68 MPa
Total Adjustment = (Weathering) x (Strike and dip orientation) x (Blasting) = (90%).(80%).(94%) = 98%

Rock Mass Strength = ((A-B)/80).0.8 . C

Where:
A = Total rating
B = Intact Rock Strength rating
C = Intact Rock Strength

Total rating = 68
Subtract IRS rating, 68 - 10 = 58
Determination of reduction factor, 58/80 = 0.72
Application of IRS, 97 x 0.72 = 69.8, MPa
Correction to 80% (insitu RMS), 69.8 x (80/100) = 55.8, MPa
Adjustment percentage = 68%
Design Rock Mass Strength (DRMS) = 55.8 x 68% = 38 MPa

Principal stresses above the crown pillar under study are expected to be as follows (Section 6.1):

Maximum principal stress, $\sigma_1$ = 46 MPa
Minimum principal stress, $\sigma_3$ = 12.2 MPa
Stress difference, $\sigma_1 - \sigma_3$ = 33.8 MPa

By using the support selection charts (Figure 2.6) the following results were obtained:

Chart (a) shows that with a maximum stress of 46 MPa and a DRMS of 38 MPa, the rock falls in zone (III) (failure controlled) and support is required to stabilise the crown pillar. Chart (c) also shows that with a stress difference of 33.8 MPa and a DRMS of
38 MPa, the rock falls in zone (III) (failure controlled) and support is required to stabilise the key blocks in the crown pillar.

Both charts indicate that spalling, rock falls, movement on joints and plastic deformation will occur in the crown pillar and rock reinforcement is required to stabilise the crown pillar (Laubscher, 1984). A support system can be chosen from chart (d) in Figure 2.6 which indicates that cable bolts, rock bolts, shotcrete and straps should be used for the support of the crown pillar. It should be noted that the support chart (d) in this system generally looks at roadway roof support and it may not be appropriate for the support of stopes and crown pillars. If the DRMS exceeds the mining environment stress which indicates no support is required, the support of key blocks which may be formed in the crown pillar should not be ignored.

(d) Application of the stability graph method

The data used for this analysis were as follows:

- Crown pillar span. = 16 m
- Crown pillar length = 40 m
- Q system value = 23.7
- Intact rock strength = 9.7 MPa
- Induced horizontal stress = 46 MPa

The following values were attributed to the different factors when determining stability number:

\[
Q' = Q \times SRF = 23.7 \times 2.5 = 59.25 \\
\text{Rock stress factor (A)} = 0.1 \\
\text{Rock defect orientation factor (B)} = 0.3 \\
\text{Design surface orientation factor (C)} = 1 \\
\text{Stability number (N)} = Q' \times A \times B \times C = 1.77 \\
\text{Shape factor (S)} = \text{Area/Perimeter of exposed surface} = 5.7
\]

The stability number (N) versus shape factor (S) was plotted on Figure 6.15. This figure shows that the crown pillar under investigation was located in a the potentially unstable zone.
(e) Results

The stability of the crown pillar at the site of investigation was assessed with RMR, Q and Laubscher Modified RMR rock mass classification systems and also the stability graph method. The results are summarised in Table 6.10.

<table>
<thead>
<tr>
<th>Rock mass classification system</th>
<th>Rating</th>
<th>Rock type</th>
<th>Stability</th>
<th>Type of support</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bieniawski's RMR system</td>
<td>67</td>
<td>Class II, good rock</td>
<td>Stable for about 2 months</td>
<td>Not required</td>
</tr>
<tr>
<td>Q system</td>
<td>23.7</td>
<td>Good rock</td>
<td>Stable</td>
<td>Not required</td>
</tr>
<tr>
<td>Modified RMR system</td>
<td>DRMS = 38</td>
<td>Good rock</td>
<td>Regular collapse or caving in the crown pillar</td>
<td>Cable bolts and rock bolts</td>
</tr>
<tr>
<td>Mathews' stability graph method</td>
<td>N= 1.77</td>
<td></td>
<td>crown pillar is located in the potentially unstable zone</td>
<td>Pillar thickness should be increased or stope height should be reduced</td>
</tr>
<tr>
<td></td>
<td>S = 5.7</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
6.5.2 Assessment of crown pillar stability using the combined empirical and analytical method

In this section the combined empirical and analytical method, as proposed in Section 3.5 is applied to the assessment of crown pillar stability. The crown pillar at the site of investigation had a 16 m span which was the limit of the stope width. Based on this 16 m span, the following section will assess the stability by comparing thickness, induced stress and rock mass strength. From this comparison a safety factor before and after the use of support can be determined. The safety factor is a direct indication of the degree of stability.

(a) Thickness estimation

The results of a computer run to determine the safe span against shear and buckling failure as a function of thickness for the conditions found at NBHC are presented in Figure 6.16. This figure, based on a modified voussoir theory (Section 3.4) shows that a 16 m wide crown pillar with a thickness less than 4 m will buckle and one with a thickness greater than this may be stable. It should be noted that if the magnitude of the confining stress is low the crown may fail in shear, and if it is too high the crown will fail in compression.

![Figure 6.16 Determination of safe span for self supporting rectangular roof versus thickness.](image)
(b) Estimation of stress

Considering the initial horizontal stress at the site (Table 6.2) and the stope height equal to 32 m the stress in the crown pillar was estimated from Figure 3.5 for different thicknesses. The results are presented in Table 6.11.

<table>
<thead>
<tr>
<th>Crown pillar thickness, m</th>
<th>Stress, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>77</td>
</tr>
<tr>
<td>8</td>
<td>62</td>
</tr>
<tr>
<td>10</td>
<td>54</td>
</tr>
<tr>
<td>12</td>
<td>48</td>
</tr>
<tr>
<td>14</td>
<td>44</td>
</tr>
<tr>
<td>16</td>
<td>41</td>
</tr>
<tr>
<td>18</td>
<td>37</td>
</tr>
<tr>
<td>20</td>
<td>33</td>
</tr>
</tbody>
</table>

(c) Estimation of pillar strength

As explained before (Section 3.3.3) the formulae which have been used for determination of pillar strength underestimate the strength of crown pillars and therefore Equation 3.15 (Section 3.3.4) was used to determine the strength of the rock mass in the crown pillar.

$$\text{RMS} = \frac{\sigma_c (\text{RMR} - \sigma_c \text{ rating})}{80} \times 0.8$$

Eq. (3.15)

Where:

- \(\text{RMS}\) = Rock Mass Strength, MPa
- \(\sigma_c\) = 97 MPa (Intact rock strength)
- \(\sigma_c\) rating = 10 (Rating for Intact rock strength)
- \(\text{RMR}\) = 68 (Rock Mass Rating)
- \(\text{RMS}\) = 56 MPa
(d) Computation of factor of safety

The factor of safety was obtained from strength/stress ratio of the pillar (Section 3.5). Figure 6.17 shows the factor of safety versus thickness of an unsupported crown pillar at the site of investigation.

![Figure 6.17](image)

**Figure 6.17** Change of the safety factor versus thickness in an unsupported crown pillar.

Using Figure 3.5 and 3.25 (Section 3.2 and Section 3.6), the minimum thickness and bolt factor required to stabilise a crown pillar with a 16 m span is shown in Table 6.12.

**Table 6.12** The minimum support required to stabilise the crown pillar of various thicknesses

<table>
<thead>
<tr>
<th>Crown pillar thickness</th>
<th>Safety factor without support</th>
<th>Safety factor BF = 1</th>
<th>Safety factor BF = 2</th>
<th>Safety factor BF = 3</th>
<th>Safety factor BF = 4</th>
<th>Safety factor BF = 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 m</td>
<td>0.74</td>
<td>0.88</td>
<td>0.94</td>
<td>0.97</td>
<td>1</td>
<td>1.07</td>
</tr>
<tr>
<td>8 m</td>
<td>0.91</td>
<td>1.06</td>
<td>1.13</td>
<td>1.17</td>
<td>1.22</td>
<td>1.3</td>
</tr>
<tr>
<td>10 m</td>
<td>1.04</td>
<td>1.25</td>
<td>1.32</td>
<td>1.38</td>
<td>1.43</td>
<td>1.52</td>
</tr>
<tr>
<td>12 m</td>
<td>1.16</td>
<td>1.39</td>
<td>1.48</td>
<td>1.53</td>
<td>1.6</td>
<td>1.7</td>
</tr>
<tr>
<td>14 m</td>
<td>1.26</td>
<td>1.51</td>
<td>1.60</td>
<td>1.67</td>
<td>1.73</td>
<td>1.85</td>
</tr>
<tr>
<td>16 m</td>
<td>1.35</td>
<td>1.62</td>
<td>1.72</td>
<td>1.79</td>
<td>1.85</td>
<td>1.98</td>
</tr>
<tr>
<td>18 m</td>
<td>1.43</td>
<td>1.71</td>
<td>1.82</td>
<td>1.89</td>
<td>2.07</td>
<td>2.10</td>
</tr>
<tr>
<td>20 m</td>
<td>1.51</td>
<td>1.8</td>
<td>1.91</td>
<td>1.99</td>
<td>2.06</td>
<td>2.19</td>
</tr>
</tbody>
</table>
It was concluded that crown pillars less than 6 metres thick even with extensive support cannot be stabilised. Crown pillars between 6 and 10 metres thick can be stabilised using bolting and crown pillars greater than 10 m thick are stable without support, although support of key blocks as always may be required.

6.5.3 Numerical analysis

The distinct element program, UDEC, was used for numerical analysis and two particular problems were analysed as follows:

i. Stress analysis
ii. Sensitivity study of the factors which affect crown pillar stability

The stress analysis was carried out to evaluate overall stability and to determine the stress in the crown pillar at the site of investigation. The second analysis involving the factors which affect crown pillar stability included a parametric study to investigate the effect different parameters have on the maximum induced stress and the safety factor.

6.5.3.1 Crown pillar modelling

No. 1 lens is one of seven stratiform ore horizons in NBHC Mine. The orebody was modelled between 950 m and 1080 m below ground surface (Level 21 and 22) in the mine. The stope height was 32 m and the crown pillar was 16 m wide. The block were created in the model by two discontinuous vertical (75°) and horizontal (15°) joint sets. Boundary stresses were assumed constant over the model.

The initial field stress values determined from tests carried out by CSIRO (Rock Stress Measurements, Mining, 1992, internal report) at 21 level (above the crown pillar) are presented in Table 6.13. It should be noted that results from the test in the country rock were affected by its nearness to stope.
Table 6.13  Measurements of stress components from overcoring method

<table>
<thead>
<tr>
<th>Test location</th>
<th>E (GPa)/ν</th>
<th>σ_{NS} (MPa)</th>
<th>σ_{EW} (MPa)</th>
<th>σ_{V} (MPa)</th>
<th>τ_{NE} (MPa)</th>
<th>τ_{EV} (MPa)</th>
<th>τ_{VN} (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>In orebody</td>
<td>79.5/0.212</td>
<td>20.6</td>
<td>37.7</td>
<td>39.9</td>
<td>-13.2</td>
<td>-0.7</td>
<td>-6.2</td>
</tr>
<tr>
<td>In country rock</td>
<td>62.3/0.138</td>
<td>36.1</td>
<td>61.1</td>
<td>57.8</td>
<td>-2.2</td>
<td>-7.8</td>
<td>-12.8</td>
</tr>
</tbody>
</table>

The rock properties are shown in Table 6.14 and using the following formulas are converted to the properties required as input data for modelling as shown in Table 6.15.

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

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

Where:
- \(K\) = Bulk modulus, GPa
- \(G\) = Shear modulus, GPa
- \(E\) = Elastic modulus, GPa
- \(\nu\) = Poisson’s ratio

Information on mechanical properties of the mine rocks which were used in the model are presented in Tables 6.15 (a) (b).

Table 6.14  Basic mechanical properties of the rocks

<table>
<thead>
<tr>
<th>Properties</th>
<th>Garnet Quartzite</th>
<th>High Grade Ore</th>
<th>Backfill material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus</td>
<td>78.5 GPa</td>
<td>62 GPa</td>
<td>0.15 GPa</td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>0.2</td>
<td>0.26</td>
<td>0.35</td>
</tr>
<tr>
<td>UCS, MPa</td>
<td>117</td>
<td>97</td>
<td>NR</td>
</tr>
</tbody>
</table>
Table 6.15 Additional properties used in modelling

<table>
<thead>
<tr>
<th>Properties</th>
<th>Garnet Quartzite</th>
<th>Orebody</th>
<th>Fill material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density Kg/m³</td>
<td>2900</td>
<td>3500</td>
<td>2300</td>
</tr>
<tr>
<td>Bulk modulus, GPa</td>
<td>43.7</td>
<td>41.4</td>
<td>0.17</td>
</tr>
<tr>
<td>Shear modulus, GPa</td>
<td>32.7</td>
<td>23.9</td>
<td>0.056</td>
</tr>
<tr>
<td>Cohesion, MPa</td>
<td>34</td>
<td>31</td>
<td>0.78</td>
</tr>
<tr>
<td>Solution condition</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

Solution condition 3 = Elasto-Plastic deformation

To determine a suitable distance for the model boundary, as described in Section 5.6.2 various boundary distance from the stope walls were examined and it was found that when the boundary is more than 6.5 stope widths away from the excavation the maximum stress in the crown pillar approaches a constant value and with increasing distance of the boundary from the excavation the change of stress in the crown pillar is negligible.

6.5.3.2 Stress analysis

In order to estimate the initial stress state for study of crown pillar between level 21 and 22 of the mine, a preliminary model was constructed. A primary model was developed based on the geological and mining parameters previously stated. Two sequences of mining were studied: First excavation of the upper 32 m stope, excavation of the lower stope (leaving the 12 metre crown pillar) and then the emplacement of backfill. The second sequence was excavation of the upper stope, fill and then excavate the lower stope. The data file input for stress analysis is shown in Appendix 3. The following two cases for the crown pillar were analysed.

1) 16 m crown pillar span with 12m thickness (both stopes open)
2) case 1 + emplacement of fill material in the upper stope

Figure 6.18 shows the schematic diagram of the model used in the study.
In both cases, cable bolt effects were not included in the input data so unstable areas were allowed to deform. Results from the models were determined by looking at five histories:

<table>
<thead>
<tr>
<th>Location of point</th>
<th>Variable</th>
</tr>
</thead>
<tbody>
<tr>
<td>History 1 throughout the model</td>
<td>Unbalanced forces</td>
</tr>
<tr>
<td>History 2 centre of the crown pillar</td>
<td>horizontal stress</td>
</tr>
<tr>
<td>History 3 centre of the crown pillar</td>
<td>Y velocity</td>
</tr>
<tr>
<td>History 4 centre of crown pillar near lower edge</td>
<td>Horizontal stress</td>
</tr>
<tr>
<td>History 5 centre of the crown pillar near lower edge</td>
<td>Y velocity</td>
</tr>
</tbody>
</table>
For case one (model with both stopes open) the following was found:

<table>
<thead>
<tr>
<th>Variable</th>
<th>Stability condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>History 1</td>
<td>Unbalanced force</td>
</tr>
<tr>
<td>History 2</td>
<td>Horizontal stress</td>
</tr>
<tr>
<td>History 3</td>
<td>Y velocity</td>
</tr>
<tr>
<td>History 4</td>
<td>Horizontal stress</td>
</tr>
<tr>
<td>History 5</td>
<td>Y velocity</td>
</tr>
</tbody>
</table>

Stability condition

- Equilibrium
- Stable
- Stable
- Zero (Isolated wedge has been formed in this part)
- Movement of isolated wedge

For case two (model with fill material in the upper stope) the following was found:

<table>
<thead>
<tr>
<th>Variable</th>
<th>Stability condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>History 1</td>
<td>Unbalanced force</td>
</tr>
<tr>
<td>History 2</td>
<td>Horizontal stress</td>
</tr>
<tr>
<td>History 3</td>
<td>Y velocity</td>
</tr>
<tr>
<td>History 4</td>
<td>Horizontal stress</td>
</tr>
<tr>
<td>History 5</td>
<td>Y velocity</td>
</tr>
</tbody>
</table>

Stability condition

- Equilibrium
- Stable
- Stable
- Zero (Isolated wedge has been formed in this part)
- Movement of isolated wedge

Figure 6.19 show the maximum and minimum principal stress in a 12 m thick crown pillar. During extraction of the upper stope stress in the pillar (where the pillar will be formed to be precise) builds up until equilibrium is reached at the end of extraction. Stress builds up again as the lower stope is excavated and stabilises at a value of 66.6 MPa. A tensile zone has also been created in the back of the lower stope which may collapse if not supported. The plastic condition in the crown pillar is shown in Figure 6.20 and the horizontal stress pattern is shown in Figure 6.21. Considering these figures it can be noticed that at the final stage of the mining sequence the wedge formed in the crown pillar has failed and support of these wedges will be required to prevent progressive failure of the crown pillar. Analysis using a Mohr-Coulomb failure criterion indicates that the remainder of the crown pillar will be stable under this conditions. Figure 6.22 shows the principal stresses in the crown pillar at the end of the mining sequence and after backfilling the upper stope. Backfilling the upper stope at this point did not significantly alter the stress values in the crown pillar, hence the stability condition of the pillar was not affected.
Figure 6.19  Principal stresses trajectories in the crown pillar at the end of the mining sequence.

**UDEC (Version 1.83)**

**LEGEND**

3/24/1994 09:49  
cycle 1000  
block plot  
principal stresses  
minimum = -6.202E+07  
maximum = 1.024E-04

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Figure 6.20 Plastic condition in the crown pillar at the end of the mining sequence.

**LEGEND**

- UDEC (Version 1.3)

- 9/09/1982 20:39
- Cycle 1000
- Block plot
- Zone total: 264
- Yielded in past (x): 170
- Tensile failure (+): 2
- Dept of Civil & Mining Engng,
  University of Wollongong
Figure 6.21 Horizontal stress pattern in the crown pillar at the end of the mining sequence.

**UDEC (Version 1.83)**

**LEGEND**

3/24/1994 09:29

cycle 1000

xx-stress contours

contour interval = 1.000E+07

min=-6.000E+07 max= 0.000E+00

-6.000E+07

-5.000E+07

-4.000E+07

-3.000E+07

-2.000E+07

-1.000E+07

0.000E+00

block plot

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Figure 6.22 Principal stresses in the crown pillar after backfilling the upper stope.

**UDEC (Version 1.83)**

**LEGEND**

3/24/1994 09:46  
cycle 1000  

block plot  
principal stresses  
minimum = -6.665E+07  
maximum = 2.106E-03

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6.5.3.3 Sensitivity study of the factors affecting crown pillar stability

In order to obtain a better understanding of the crown pillar failure mechanism, a sensitivity study was carried out using the UDEC program. In the analysis model of crown pillar under study was used to investigate the effect various parameters have on stability in the crown pillar. For a particular series of models all but one of the following parameters; joint spacing, joint friction angle, orebody cohesion, elastic modulus and horizontal stress were kept constant so that the effect of each individual parameter could be monitored. the following values were taken as the base:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus</td>
<td>62 GPa</td>
</tr>
<tr>
<td>Joint friction angle</td>
<td>31°</td>
</tr>
<tr>
<td>Orebody cohesion</td>
<td>31 MPa</td>
</tr>
<tr>
<td>Country rock cohesion</td>
<td>34 MPa</td>
</tr>
<tr>
<td>Orebody density</td>
<td>3500 kg/m³</td>
</tr>
<tr>
<td>Country rock density</td>
<td>2900 kg/m³</td>
</tr>
<tr>
<td>Insitu horizontal stress</td>
<td>28.7 GPa</td>
</tr>
<tr>
<td>Vertical stress</td>
<td>16 GPa</td>
</tr>
<tr>
<td>Joint normal stiffness</td>
<td>50 GPa/m</td>
</tr>
<tr>
<td>Joint shear stiffness</td>
<td>5 GPa/m</td>
</tr>
</tbody>
</table>

Figure 6.18 shows the geometry of the stope and the crown pillar which was modelled. In all cases the dimensions of the stope and crown pillar were as follows:

- Stope height 32 m,
- Crown pillar width 16 m, and
- Crown pillar thickness 12 m.

(a) Effect of vertical joint spacing

The results of this study are shown in Figures 6.23 and 6.24 and Table 6.16. These results indicates that when joint spacing decreases stress in the crown pillar decreases and block movement in the crown pillar and stope walls also increase. The results also show (Figure 6.24) that the factor of safety against failure decreases with decreasing joint spacing.
Figure 6.23 Maximum horizontal stress versus vertical joint spacing.

Figure 6.24 Factor of safety versus vertical joint spacing.
Table 6.16 Summary of the analysis for different vertical joint spacing.

<table>
<thead>
<tr>
<th>Vertical joint spacing, metre</th>
<th>Horizontal stress in centre of crown pillar, MPa</th>
<th>Max. shear contour, MPa</th>
<th>Safety factor in the crown pillar</th>
<th>Overall condition of the crown pillar</th>
<th>Stope walls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>62.6</td>
<td>60</td>
<td>1.5</td>
<td>Crown pillar is stable, only wedge failure,</td>
<td>Movement of stope walls towards each other, Max = 60 mm</td>
</tr>
<tr>
<td>6</td>
<td>62</td>
<td>60</td>
<td>1.5</td>
<td>Crown pillar is stable, only wedge failure,</td>
<td>Movement of stope walls towards each other, Max = 61 mm</td>
</tr>
<tr>
<td>4</td>
<td>61</td>
<td>60</td>
<td>1.3</td>
<td>Crown pillar at yield surface and wedge failure</td>
<td>Movement of stope walls towards each other, Max = 72 mm</td>
</tr>
<tr>
<td>3</td>
<td>52</td>
<td>50</td>
<td>1.2</td>
<td>Crown pillar at yield surface and wedge failure</td>
<td>Movement of stope walls towards each other, Max=101 mm</td>
</tr>
<tr>
<td>2</td>
<td>52</td>
<td>50</td>
<td>1.2</td>
<td>increasing shear movement in the crown pillar</td>
<td>Movement of stope walls towards each other, Max=110 mm</td>
</tr>
<tr>
<td>1</td>
<td>50</td>
<td>50</td>
<td>1.1</td>
<td>increasing shear movement, crown pillar at yield surface</td>
<td>Instability increased in the stope walls, Max=117 mm</td>
</tr>
</tbody>
</table>
(b) Effect of horizontal stress

The effect of varying horizontal stress on the stability of the crown pillar was examined by changing different values of insitu horizontal stress. The results are presented in Figures 6.25 (a) and (b) and Table 6.17.

![Figure 6.25 (a) Maximum horizontal stress versus insitu horizontal stress.](image)

Figure 6.25 (b) shows that with increasing the horizontal stress the safety factor decreases and when the insitu horizontal stress is greater than 56 MPa failure will occur in the crown pillar.

![Figure 6.25 (b) Factor of safety versus insitu horizontal stress.](image)
The variation of insitu horizontal stress indicate that the maximum horizontal stress in the crown pillar increases from 43 MPa to 96 MPa and the factor of safety decreases from 2.2 to less than 1. These results suggest that for ±10 % variation of insitu horizontal stress the change of maximum horizontal stress and factor of safety in the pillar are 4.5 % and 2.5 % respectively.

**Table 6.17 Summary of the analysis for different insitu horizontal stresses**

<table>
<thead>
<tr>
<th>Insitu horizontal stress, MPa</th>
<th>Max horizontal stress, MPa</th>
<th>Horizontal stress in centre of crown pillar, MPa</th>
<th>Max shear stress, MPa</th>
<th>Safety factor the in crown pillar</th>
<th>Overall condition of the crown pillar</th>
<th>Stope sidewalls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>43</td>
<td>44</td>
<td>40</td>
<td>2.2</td>
<td>Crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 41 mm</td>
</tr>
<tr>
<td>13.2</td>
<td>46.7</td>
<td>46.7</td>
<td>40</td>
<td>1.8</td>
<td>Crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 44 mm</td>
</tr>
<tr>
<td>20</td>
<td>64.35</td>
<td>62.6</td>
<td>60</td>
<td>1.8</td>
<td>Crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 60 mm</td>
</tr>
<tr>
<td>40</td>
<td>73.2</td>
<td>73</td>
<td>60</td>
<td>1.5</td>
<td>Crown pillar in plastic condition</td>
<td>Some part of stope walls has failed</td>
</tr>
<tr>
<td>50</td>
<td>96</td>
<td>-</td>
<td>-</td>
<td>1.1</td>
<td>Crown pillar in plastic condition</td>
<td>Some part of stope walls has failed</td>
</tr>
<tr>
<td>56</td>
<td>81</td>
<td>-</td>
<td>-</td>
<td>0.8</td>
<td>failed</td>
<td>-</td>
</tr>
</tbody>
</table>

(c) Effect of elastic modulus of orebody

The effect of the change of the orebody elastic modulus on crown pillar stability is presented in Table 6.18 and Figures 6.26 (a) and ((b). The results show that for a 10 % variation of this parameter the change of the maximum horizontal stress and factor of safety of the crown pillar in the model are 0.01 % and 0.5 % receptively.
Table 6.18 Summary of analysis for change of elastic modulus in orebody

<table>
<thead>
<tr>
<th>Elastic modulus GPa</th>
<th>Max horizontal stress, MPa</th>
<th>Horizontal stress in centre of crown pillar, MPa</th>
<th>Max shear stress, MPa</th>
<th>Factor of Safety in the crown pillar</th>
<th>Overall condition of the crown pillar</th>
<th>Stope sidewalls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>E = 10</td>
<td>60.8</td>
<td>54.4</td>
<td>50</td>
<td>1.7</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls increased Max = 202 mm</td>
</tr>
<tr>
<td>E = 20</td>
<td>62.3</td>
<td>56.9</td>
<td>55</td>
<td>1.7</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 112 mm</td>
</tr>
<tr>
<td>E = 40</td>
<td>64</td>
<td>58.8</td>
<td>55</td>
<td>1.6</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 85 mm</td>
</tr>
<tr>
<td>E = 50</td>
<td>64.35</td>
<td>63.3</td>
<td>60</td>
<td>1.5</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 61 mm</td>
</tr>
<tr>
<td>E = 60</td>
<td>64.35</td>
<td>63.3</td>
<td>60</td>
<td>1.5</td>
<td>Crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 60 mm</td>
</tr>
<tr>
<td>E = 80</td>
<td>65</td>
<td>63.2</td>
<td>60</td>
<td>1.4</td>
<td>Crown pillar is stable</td>
<td>Stope walls convergence Max = 58 mm</td>
</tr>
<tr>
<td>E = 100</td>
<td>65</td>
<td>63.3</td>
<td>60</td>
<td>1.4</td>
<td>Crown pillar is stable</td>
<td>Stope walls convergence Max = 58 mm</td>
</tr>
</tbody>
</table>
Figure 6.26 (a) Maximum horizontal stress versus elastic modulus.

Figure 6.26 (b) Factor of safety versus elastic modulus.
(e) Effect of orebody cohesion

The results of the analysis of variation of orebody cohesion are presented in Table 6.19 and Figures 6.27 (a) and (b). Figure 6.27 (b) shows that when the cohesion of the rock material increases, the stress level in the crown increases and the safety factor also increases.

Table 6.19 Summary of the analysis for different orebody cohesion

<table>
<thead>
<tr>
<th>Cohesion MPa</th>
<th>Max. horizontal stress, MPa</th>
<th>Horizontal stress in centre of crown pillar, MPa</th>
<th>Max shear contour in crown pillar, MPa</th>
<th>Safety factor in the crown pillar</th>
<th>Overall condition of the crown pillar</th>
<th>Stope sidewalls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>C = 70</td>
<td>68.1</td>
<td>63.1</td>
<td>60</td>
<td>2</td>
<td>No significant movement, crown pillar is stable</td>
<td>Stope walls convergence, Max = 60 mm</td>
</tr>
<tr>
<td>C = 60</td>
<td>68.1</td>
<td>63.1</td>
<td>60</td>
<td>1.8</td>
<td>No significant movement, crown pillar is stable</td>
<td>Stope walls convergence, Max = 60 mm</td>
</tr>
<tr>
<td>C = 50</td>
<td>68</td>
<td>63.5</td>
<td>60</td>
<td>1.8</td>
<td>No significant movement, crown pillar is stable</td>
<td>Stope walls convergence, Max = 60 mm</td>
</tr>
<tr>
<td>C = 40</td>
<td>68</td>
<td>63.5</td>
<td>60</td>
<td>1.5</td>
<td>No significant movement, crown pillar is stable</td>
<td>Stope walls convergence, Max = 60 mm</td>
</tr>
<tr>
<td>C = 30</td>
<td>64</td>
<td>62</td>
<td>60</td>
<td>1.4</td>
<td>No significant movement, crown pillar is stable</td>
<td>Stope walls convergence, Max = 60 mm</td>
</tr>
<tr>
<td>C = 20</td>
<td>47</td>
<td>44</td>
<td>40</td>
<td>1.2</td>
<td>Some parts of crown pillar at yielding condition</td>
<td>Stope walls convergence, Max = 60 mm</td>
</tr>
<tr>
<td>C = 10</td>
<td>46.7</td>
<td>44</td>
<td>40</td>
<td>1.1</td>
<td>Crown pillar at yielding condition</td>
<td>Stope walls convergence, Max = 60 mm</td>
</tr>
</tbody>
</table>
The results show that for $\pm 10\%$ variation of cohesion the change of maximum stress and factor of safety in the crown pillar are 2.2\% and 1.7\% respectively.

**Figure 6.27 (a)** Maximum horizontal stress versus orebody cohesion.

**Figure 6.27 (b)** Factor of safety versus orebody cohesion.
(f) Effect of the joint friction angle

The results of the analysis for various joint friction angles are presented in Figures 6.28 (a) and (b) and Table 6.20. These results indicate that the rock mass in the crown pillar has yielded for friction angles less than 20° and failure occurs when the friction angle is less than 15°.

![Figure 6.28 (a) Maximum horizontal stress versus joint friction angle.](image)

![Figure 6.28 (b) Safety factor versus joint friction angle.](image)
The results also show that for ± 10% variation of this parameter the change of the maximum horizontal stress and factor of safety in the crown pillar are 1.1% and 3%.

Table 6.20 Summary of the analysis for variation of joint friction angle

<table>
<thead>
<tr>
<th>Joint friction angle</th>
<th>Max. horizontal stress, MPa</th>
<th>Horizontal stress in centre of crown pillar, MPa</th>
<th>Max shear stress in the crown pillar, MPa</th>
<th>Safety factor in the crown pillar</th>
<th>Overall condition of the crown pillar</th>
<th>Stope sidewalls movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi = 50^\circ$</td>
<td>66.6</td>
<td>58.6</td>
<td>60</td>
<td>1.6</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 51 mm</td>
</tr>
<tr>
<td>$\phi = 45^\circ$</td>
<td>66.2</td>
<td>62</td>
<td>60</td>
<td>1.6</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 50 mm</td>
</tr>
<tr>
<td>$\phi = 40^\circ$</td>
<td>64.35</td>
<td>62.8</td>
<td>60</td>
<td>1.5</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 50 mm</td>
</tr>
<tr>
<td>$\phi = 30^\circ$</td>
<td>64.4</td>
<td>63.3</td>
<td>60</td>
<td>1.5</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 61 mm</td>
</tr>
<tr>
<td>$\phi = 25^\circ$</td>
<td>64</td>
<td>63</td>
<td>60</td>
<td>1.4</td>
<td>Plastic increased in crown pillar, stable</td>
<td>Movement of stope walls towards each other, Max = 60 mm</td>
</tr>
<tr>
<td>$\phi = 20^\circ$</td>
<td>64</td>
<td>63</td>
<td>60</td>
<td>1.2</td>
<td>Crown pillar at yield surface</td>
<td>Instability increased in stope wall which is with dip.</td>
</tr>
<tr>
<td>$\phi = 15^\circ$</td>
<td>64</td>
<td>61</td>
<td>60</td>
<td>1</td>
<td>Significant movement, crown pillar is unstable</td>
<td>Failure in some part of stope walls</td>
</tr>
<tr>
<td>$\phi = 10^\circ$</td>
<td>-</td>
<td>44</td>
<td>-</td>
<td>-</td>
<td>failed</td>
<td>failed</td>
</tr>
</tbody>
</table>
(g) Effect of crown pillar thickness on stability

A series of models were developed (based on the primary model in Figure 6.18) to study the effect of the thickness of the pillar on factor of safety, whilst keeping all other parameters constant. A summary of this analysis is presented in Tables 6.21 and Figures 6.29 (a) and (b).

**Table 6.21** Summary of the analysis for different crown pillar thickness

<table>
<thead>
<tr>
<th>Crown pillar thickness, metre</th>
<th>Max horizontal stress in the crown pillar, MPa</th>
<th>Average horizontal stress in the crown pillar, MPa</th>
<th>Safety factor in the crown pillar</th>
<th>Overall condition of the crown pillar</th>
<th>Stope wall movements</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>53</td>
<td>43.5</td>
<td>1.7</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 52 mm</td>
</tr>
<tr>
<td>18</td>
<td>57.2</td>
<td>45</td>
<td>1.6</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 50 mm</td>
</tr>
<tr>
<td>16</td>
<td>58</td>
<td>47</td>
<td>1.5</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 54 mm</td>
</tr>
<tr>
<td>14</td>
<td>59.1</td>
<td>48</td>
<td>1.4</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max=61.3 mm</td>
</tr>
<tr>
<td>12</td>
<td>64.35</td>
<td>49</td>
<td>1.3</td>
<td>No significant movement, crown pillar is stable</td>
<td>Movement of stope walls towards each other, Max = 61 mm</td>
</tr>
<tr>
<td>10</td>
<td>66.9</td>
<td>55</td>
<td>1.2</td>
<td>Some part of the crown pillar at yielding condition</td>
<td>Instability increased in stope walls</td>
</tr>
<tr>
<td>8</td>
<td>67</td>
<td>61.7</td>
<td>1.06</td>
<td>Crown pillar at yielding condition</td>
<td>Failure in some part of stope walls</td>
</tr>
<tr>
<td>7</td>
<td>Failed</td>
<td>-</td>
<td>0.8</td>
<td>Failed</td>
<td>Failure in some part of stope walls</td>
</tr>
</tbody>
</table>
The results show that a change in the crown pillar thickness has a significant influence on the stability of the crown pillar. Increasing the thickness makes the crown pillar more stable and reducing the thickness causes instability. Stable conditions were noticed when the thickness was greater than 12 m. Between 7 m and 12 m the pillars were in a plastic condition. Failure occurred when the thickness was less than 7 m.

Figure 6.29 (a) Maximum horizontal stress versus crown pillar thickness

Figure 6.29 (b) Safety factor versus crown pillar thickness.
6.6 Discussion

Three methods of rock mass classification ie. RMR, Q, and Modified RMR system in addition to the stability graph method were used to assess crown pillar stability. Bieniawski’s RMR system predicts that the crown pillar under investigation would have been stable for about 2 months and no support would be required. In reality the time to extract and fill a stope is longer than this period, therefore, the RMR system suggests that the crown pillar should be supported to enable it to remain stable during the life of the stope. This agrees with the conditions found at the mine.

Based on the analysis by the Q-system the crown pillar is stable and support will not be required. However support is recommended because the value obtained from this analysis same as CSA Mine is very close to the border line of unstable zones.

The results of analysis by Modified RMR-system indicates crown pillar is located in unstable zone and it should be supported to be stable. Instability in the crown pillar is due to high horizontal stress and large differences between the major and minor principal stresses.

Analysis by the stability graph method indicates that the crown pillar is located in the potentially unstable zone. Potvin’s method (Figure 2.25) suggests that using a bolt factor of about 7 metres of cable / m² stabilises the crown pillar at the mine. In comparison with field observation this method overestimates the support required for stabilising the crown pillar in this mine.

The results obtained from analysis by the combined empirical and theoretical method shows that a 16 m wide, 12 m thick crown pillar is stable without support. However in this case the safety factor is only 1.16 which is relatively low and may cause problems for long term stability. Using different bolt factors improves the factor of safety in the pillar. Table 6.12 shows the change of factor of safety in a 12 m thick crown pillar when bolts are installed. In comparison with the field using a bolt factor of about 2 metres of cable bolt/m² is comparable with reality and gives a safety factor of about 1.5. Using a bolt factor of more than 2 metres of cable bolt/m² is conservative and less than 2 metres of cable bolt/m² would not be adequate in this case. Generally the crown pillars at the field are supported with 6 to 8 cable bolts of between 7 m and 10 m at a spacing of 2 m.
The method shows that pillars with 6 and 10 m thick which would fail without support, can be stabilised by bolting. The method also suggest that instead of increasing the thickness, the pillar can be bolted to obtain the same level of factor of safety. For example when the thickness increases from 12 m to 18 metres (Table 6.13), the factor of safety of an unsupported pillar changes from 1.16 to 1.5 while supporting a 12 m thick crown pillar with a bolt factor of about 2 metres of cable bolt/m², increases the factor of safety to the same value ie 1.5. As bolts are used to stabilise isolated wedges it would seem more practical to increase the safety factor using bolts rather than increase the thickness (obviously within limits).

Comparing the results obtained from this method with the results of numerical analysis shows that both methods predict that crown pillars greater than 12 m thick are stable. The combined empirical and theoretical method suggests that crown pillars between 8 m and 10 m thick are stable only if supported, the results of UDEC analysis also show that pillars between 8 m and 12 m are in a yielding condition and they should be supported to maintain stability. Both methods indicate that pillars less than 6 m fail in this condition. The factors of safety of various thickness of crown pillar obtained from both methods are shown in Figure 6.30 and it can be seen that the results are comparable.

Figure 6.30 Safety factor versus crown pillar thickness.
A summary of the results of the sensitivity study are shown in Table 6.22, in order of the effect of each the parameter on stability. Variation of all parameters which were used in sensitivity study can affect the maximum horizontal stress and the stability condition of the crown pillar, however, the levels of their effect are different. Table 6.22 indicates that an error in the measurement of joint friction angle and initial horizontal stress of ±10% will have significant effect on the stability while the same percentage error in measurement of elastic modulus will not significantly influence the results of the analysis. Accurate measurement of cohesion is also relatively more important than elastic modulus for prediction of stability condition.

Table 6.22 Results of sensitivity study of the crown pillar

<table>
<thead>
<tr>
<th>Variation of parameter (±10%)</th>
<th>Change of factor of safety in the crown pillar(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint friction angle</td>
<td>3</td>
</tr>
<tr>
<td>In situ horizontal stress</td>
<td>2.5</td>
</tr>
<tr>
<td>Orebody cohesion</td>
<td>1.7</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>0.5</td>
</tr>
</tbody>
</table>

6.7 Conclusion

The followings are the most important conclusion of this study:

- The RMR-System indicates that support would be required if the crown pillar was to remain stable during the life of the stope. The Q-System suggests that no support would be required. The MRMR-System seemed to be a bit conservative when compared with reality. Mathews stability graph method was conservative in this cases and overestimates the support required for stabilising the crown pillar at this mine.

- UDEC program indicates that crown pillars more than 12 m thick are generally stable and crown pillars of 7 m to 12 m thick are in a yielding condition and should be supported to maintain stability. Crown pillars less than 7 m fail under these conditions.
• Numerical study also shows that backfilling the upper stope in case of 12 metre thick crown pillar (which remains stable), had no significant effect on stress redistribution. However it helps stabilise the stope walls by limiting movement.

• Results of sensitivity study shows that the parameters which affect stability in the crown pillar in order of their importance are joint friction angle, initial horizontal stress and cohesion. Elastic modulus had little influence on stability. Increasing the thickness makes the crown pillar more stable and reducing the joint spacing cause the increase of blocks movements in the crown pillar and decreases the factor of safety.

• Field observations at NHBC Mine show that crown pillars with 16 m span and 12 m thickness are generally stable with the help 6 to 8 cable bolts of 7 to 10 m. The results predicted by numerical analysis indicated that crown pillars greater than 12 m thick are stable without support. However, although a compression arch is formed in crown pillars greater than 12 m thick, a tension zone is created which may lead to overbreak in the back of the stope and possibly progressive failure of the pillar. Therefore, a support system in this situation would be required to resist shear and tensile forces in the joints, thus locking in key blocks, thereby helping the pillar to support itself and accept the redistributed stresses.

• When the combined empirical and theoretical method was applied to NBHC Mine, the method satisfactorily predicted the crown pillar geometry at the site of investigation and showed that a 12 m thick crown pillar is stable without support. However because the safety factor is relatively low it may cause stability problems in the long term. As a result support is required to improve the factor of safety in the crown pillar for longer term stability. Using a bolt factor of about 2 metres of cable bolt/m² is enough to maintain the stability for the life of the stope, which compares favourably with that found at the mine.

• The results of the analysis by this method also show that crown pillars greater than 10 m thick are stable without support and failure occurs when the thickness is less than 10 m. The method indicates that bolting improves the factor of safety in pillars, and pillars between 6 and 10 m thick can be stabilised by bolting. The method also suggests that instead of increasing the thickness, the pillar can be bolted to obtain the same level of factor of safety. It should be noted that support of key blocks is required even when the pillar is thick enough to remain stable by itself, therefore, using a thinner pillar with bolting seems to be more economic.
Comparison of the results obtained from the combined empirical and theoretical method with the results of numerical method indicates that this method is useful for initial estimation of the geometry and evaluation of stability of crown pillars. The safety factors obtained from both methods for different geometries of crown pillar are comparable.
CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

7.1 Conclusion

This thesis has described the methods and results of analytical and numerical investigations into the design and stability assessment of crown pillars in two metalliferous mines. Available theoretical and empirical design methods were used to evaluate the behaviour of the rock mass in the crown pillar during mining activities. Laboratory tests carried out on rock samples collected from the mines were used to help derive the characteristics of the rock mass. The results of these tests were used in numerical as well as empirical and theoretical methods of design.

The most obvious conclusion from this work is that no individual method or system is adequate to determine the required geometry of a crown pillar in a given rock mass. Any of the rock mass classification systems described can be used to gain an initial understanding of the capabilities of the rock mass constituting the crown pillar. These systems also give an indication of support requirements, although the support specification charts based on these systems are generally for roadways. The major weakness of all of the classification systems reviewed was that although they propose support requirements for a given span, they do not give any indication of required
thickness or the relationship between span and maximum strike length. Nor do they give any indication of the support required for stabilising blocks formed by the intersection of discontinuities.

To determine if support of key blocks or individual rock wedges is required, it is necessary to conduct a discontinuity survey. This survey may take the form of analysis of borehole cores, but preferably it should be a scanline survey underground and as near possible the area where the crown pillar will be formed. The data from the discontinuity survey is reduced to determine the shape, size and stability condition of wedges formed in the crown pillar. From this information initial support requirements can be calculated.

To overcome the general limitation that rock mass classification systems are unidimensional, it is advisable to link the system with some method which takes into account the other two dimensions. Matthews stability graph method, the basis of which is a modified Q value, links the two horizontal dimensions of the crown pillar by use of the hydraulic radius of the plane. To relate span and thickness, a first estimate of requirements can be gained by using a modified voussoir theory, which relates thickness and span to the potential for failure under a given stress regime. After the first pass at determining geometry it is necessary to determine the stress level in the pillar for that geometry.

Two methods of determining the stress level in the pillar are; the modified tributary area theory and numerical analysis using the distinct element code UDEC. For simple situations, the tributary area theory is adequate and results compare favourably with numerical analysis. It is envisaged, however, that the tributary area method would be too cumbersome in more complex situations, and as such numerical analysis is recommended for such occasions. UDEC also has the advantage that it allows "what if" studies to be conducted.

These 'what if' studies allow the designer to determine what effect various parameters, such as modulus, joint stiffness, cohesion etc., have on crown pillar stability. The other benefit from these parametric studies is that they can indicate the affect that data accuracy can have on the results from numerical models. It was shown, for example, that although a 10% variation in virgin stress level, caused a 5% variation in the stress level predicted in a particular crown pillar, a 10% variation in cohesion produced only a 2.2% increase in crown pillar stress. Other conclusions from these studies were:
• Increasing the thickness makes the crown pillar more stable, not so much by increasing the strength of the pillar but by reducing the maximum stress.

• Under different joint spacings and orientations the change of the maximum stress and safety factor in the crown pillar was considerable for the same properties of rock.

• When a structural weakness zone cuts across the crown pillar it may control the mode of failure. But when there is no significant structural weakness in the crown pillar, high horizontal stress is the major factor that controls the onset of failure. This can be stated without consideration of the failure mode, but the various geological features control how the failure will develop.

• The stability of the crown pillar and the stope walls can not be considered independently, as instability in either can lead to instability in the other and ultimately to global instability.

Once the stress level in the proposed pillar has been determined it is then necessary to determine the pillar strength and, by relating this to stress, determine the factor of safety of the pillar. Pillar strength can be calculated using any number of empirical methods, however, care should be taken when using the coal based formulae as these tend to underestimate pillar strength. This thesis used the rock mass strength rating (which can be related to RMR) to determine rock mass strength. The results using this criterion were more in line with reality than the results using traditional pillar formulae. Once the factor of safety has been calculated, it can be determined whether the pillar is stable or unstable.

The value which is used to define the boundary between stable and unstable can include an allowance for additional safety, but the base value is unity. If the pillar is stable, the discontinuity structure should be analysed to determine if isolated wedges have been formed as discussed previously. Support should be recommended if required and the final design proposed. If the pillar has been deemed to be unstable then reinforcement may be required.

Reinforcement in the form of cable bolts can be determined using Potvin's bolting factor chart. This chart indicates the apparent increase in the rock mass rating, and therefore pillar strength, which will be achieved by using a particular bolt factor (metres of cable/m²). The 'new strength' of the pillar can then be determined and compared with the calculated stress level. If the pillar can not be stabilised using cables (which may also be necessary to stabilise individual wedges) it will be necessary to increase the pillar
thickness and recalculate the strength and iterate to the final design. The complete methodology for design is shown schematically in figure 7.1.

Figure 7.1 Methodology for design of a crown pillar
7.2 Recommendations for future research

This thesis has attempted to produce a basic methodology for the design of crown pillars in metalliferous mines. To improve the methodology and determine its applicability to design of crown pillar in all situations it is recommended that the number of case studies be substantially increased. In particular, it is important to gather and analyse data from failed pillars to determine if the system would have predicted failure, and if so, predict what measures if any could have been taken to prevent (or postpone) failure.

It is also recommended that work be done to determine if current methods of calculating the strength of pillars in metal mines are accurate and reliable; most work to date has been for coal mines and the formulae underestimate the strength of metal mine pillars.
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APPENDIX ONE

COMPUTER PROGRAMS FOR SOLUTION OF SEQUENTIAL EQUATIONS FOR ROCK BEAMS, SQUARE AND RECTANGULAR PLATES BASED ON VOUSSOIR BEAM THEORY
PROGRAM FOR DETERMINATION OF MAXIMUM SAFE SPAN OF A ROOF BEAM BASED ON VOUSSOIR ARCH THEORY

REAL N,L,N1
DATA G,T,SO,N/
DO 450 FC=5,50,5
Z0=T*(1-((2./3.)*N))
100 A=(4*N*FC)/G
C1=A*A*0.1718*FC/E
C2=(A*A*Z0*Z0)*(1-((11/24)*(FC/E)))
Y=C1*C1+4*C2
IF (Y.LT.0.0) GO TO 500
IF(SQRT(Y).GT.C1)THEN
S1=(SQRT(Y)-C1)/2.
S=SQRT(S1)
ELSE
GO TO 500
END IF
IF(ABS(S-S0).LE.0.01)GO TO 400
L=S+(8.*Z0*Z0)/(3.0*S)
DL=(11./24.)*(FC/E)*L
X=(3*S/8.)*(((8.*Z0*Z0)/(3*S))-DL)
IF(X.LT.0.0)GO TO 400
Z=SQRT(X)
N1=(3./2.)*(1.-((Z/T)))
IF(N1.GT.1.OR.N1.LT.0.0)GO TO 400
N=N1
Z0=Z
S0=S
GO TO 100
400 WRITE(*,*) S, FC, Z, T/S
450 CONTINUE
500 STOP
END
PROGRAM FOR DETERMINATION OF ROOF BEAM
FACTOR OF SAFETY AGAINST SHEAR AND
COMPRESSION
IMPLICIT DOUBLE PRECISION (A-Z)
REAL N,K,L,N1
DATA G,UCS,PHI/
DO 600 E=20000,100000,10000
FC1=0.0
DO 560 T=1,21,2
N=0.75
DO 550 S=1,100,1
Z=T*(1.-2.*N/3.)
50 FC=(S*S*G)/(4.*N*Z)
FAV=0.5*FC*((2./3.)+0.5*N)
L=S+8.*Z*Z/(3.*S)
DL=(FAV*L)/E
A=((3.*S)/8.*)((8.*Z*Z)/(3.*S))-DL
IF(A.LT.0.0)THEN
WRITE(*,*) A,S
GO TO 560
END IF
Z1=DSQRT(A)
N1=1.5*(1.-Z1/T)
IF(DABS(FC-FC1).GT.0.000001)THEN
FC1=FC
N=N1
Z=Z1
GO TO 50
END IF
FSS=(FC)*N*DTAN(PHI)/(S*G)
FSC=UCS/(FC)
WRITE(*,*) FSS,S,T
550 CONTINUE
560 CONTINUE
600 CONTINUE
STOP
END
PROGRAM FOR DETERMINATION OF SQUARE ROOF FACTOR OF SAFETY AGAINST SHEAR AND COMPRESSION

IMPLICIT DOUBLE PRECISION (A-Z)
REAL N,K,L,N1
DATA G,V,UCS,PHI/0.03,.25,60.,.52/
DO 600 E=20000,120000,10000
FC1=0.0
DO 560 T=1,21,2
N=0.75
DO 550 S=1,150,1
Z=T*(1-2.*N/3.)
50 FC=(S*S*G)/(12.*N*Z)
FAV=0.5*FC*(2./3.)+0.5*N)
L=S+8.*Z*Z/(3.*S)
DL=(FAV*L*(1-V))/E
A=((3.*S)/8.)*(((8.*Z*Z)/(3.*S))-DL)
IF(A.LT.0.0)THEN
WRITE(*,*) A,S
GO TO 560
END IF
Z1=DSQRT(A)
N1=1.5*(1.-Z1/T))
IF(ABS(FC-FC1).GT.0.000001)THEN
FC1=FC
N=N1
Z=Z1
GO TO 50
END IF
FSS=2.*FC*N*TAN(PHI)/(S*G)
FSC=UCS/(FC)
WRITE(*,*) FSS,S,T
550 CONTINUE
560 CONTINUE
600 CONTINUE
STOP
END
PROGRAM FOR DETERMINATION OF
RECTANGULAR ROOF FACTOR OF SAFETY
AGAINST SHEAR AND COMPRESSION

REAL N, K, L, N1
DATA G, V, UCS, PHI, B/
DO 600 E = 20000, 120000, 10000
FC1 = 0.0
DO 560 T = 1, 21, 2
N = 0.75
DO 550 S = 1, 150, 1
Z = T * (1. - (2. * N/3.))
K = S/B
Y = .5 * S * ((SQRT(K * K + 3) - K)
50   FC = (S * S * G) * (.25 - (Y * K/(3 * S)))/(N * Z)
FAV = (7./12.) * K * FC
L = S + 8.*Z*Z/(3.*S)
DL = (FAV*L*1.-(K*V))/E
A = ((3.*S)/8.)*(((8.*Z*Z)/(3.*S))-DL)
IF(A.LT.0.0) THEN
WRITE(*,*) A, S
GO TO 560
END IF
Z1 = DSQRT(A)
N1 = 1.5*(1.-Z1/T))
IF(ABS(FC-FC1).GT.0.000001) THEN
FC1 = FC
N = N1
Z = Z1
GO TO 50
END IF
FSS = FC*N*B*TAN(PHI)/(G*S*(B-Y)
FSC = UCS/(FC)
WRITE(*,*) FSS, S, T
550 CONTINUE
560 CONTINUE
600 CONTINUE
STOP
END
APPENDIX TWO

DATA FILE INPUT FOR STRESS ANALYSIS AND PARAMETRIC STUDY OF CROWN PILLAR IN CSA MINE
Data file input for stress analysis of crown pillar in CSA Mine.

set degree off
block -60 0 -60 145 192 145 192 0
round 0.05
change mat=2 con=3
jreg -60 0 -60 145 192 145 192 0
Jset -80 0 200 0 0 0 12 0 0 0
change 60 72 65 80 mat=3 con=3
jreg -60 0 -60 145 192 145 192 0
jset 0 0 300 0 0 0 5 0 0 0
save b81.sav
gen edge 20
bound str -28.6e6 0 -16e6
insitu str -28.2e6 0 -16e6 szz -18e6
grav 0 -10
prop mat=1 d=2900 k=28e9 g=26e9
prop mat=1 jkn=50e9 jks=5e9 jf=.6
prop mat=1 kn=50e9 ks=5e9 jf=.6
prop mat=5 d=2900 k=28e9 g=26e9
prop mat=5 jkn=50e9 jks=5e9 jf=.5
prop mat=5 kn=50e9 ks=5e9 jf=.5
change jcon=2 jmat=1
* cohesion(ore)=50, country=34
prop mat=2 d=2900 k=39e9 g=23e9 coh=34e6
* for E=85 GPa
prop mat=3 den=3500 k=57e9 g=34e9 coh=50e6
damp auto
be gen -61 193 -1 146
be mat=5
be fix 0 -300 300 0
be stiff
change jmat=1
hist unbal
cyc 150
save b81.sav
*stop
pr max
reset hist
reset dis
change 58 68 80 100 con=0
change 55 65 100 120 con=0
change reg 59 80 52 120 64.5 120 71 80 con=0
hist unbal
hist n=50 sxx 69 67 ty 1
hist ydis 69 67
hist yvel 69 67
hist sxx 71 71
hist ydis 71 71
hist yvel 71 71
cyc 500
pr max
save b84.sav
del 65 75 45 65
del 68 78 25 45
hist unbal
hist n=50 sxx 69 67 ty 1
hist ydis 69 67
hist yvel 69 67
hist sxx 71 71
hist ydis 71 71
hist yvel 71 71
cyc 500
pr max
save b85.sav
*stop
restart b84.sav
prop mat=4 d=2300 k=.17e6 g=.056e6 coh=.75e6
change 58 68 80 100 mat=4 con=3
change 55 65 100 120 mat=4 con=3
hist unbal
hist n=50 sxx 69 67 ty 1
hist ydis 69 67
hist yvel 69 67
hist sxx 71 71
hist ydis 71 71
hist yvel 71 71
hist sxx 63 68
hist ydis 63 68
hist yvel 63 68
hist sxx 66 73
hist ydis 66 73
hist yvel 66 73
hist sxx 66 77
hist ydis 66 77
hist yvel 66 77
cyc 400
pr max
save b84f.sav
del 65 75 45 65
del 68 78 25 45
hist unbal
hist n=50 sxx 69 67 ty 1
hist ydis 69 67
hist yvel 69 67
hist sxx 71 71
hist ydis 71 71
hist yvel 71 71
hist sxx 63 68
hist ydis 63 68
hist yvel 63 68
hist sxx 66 73
hist ydis 66 73
hist yvel 66 73
hist sxx 66 77
hist ydis 66 77
hist yvel 66 77
cyc 500
pr max
save b85f.sav
stop
APPENDIX THREE

DATA FILE INPUT FOR STRESS ANALYSIS AND PARAMETRIC STUDY OF CROWN PILLAR IN NBHC MINE
Data file input for stress analysis of crown pillar in NBHC Mine.

set degree off
round 0.1
block -48 0 -48 128 160 128 160 0
jreg -48 0 -48 128 160 128 160 0
Jset 75 0 300 0 0 0 8 0 0 0
jreg -48 0 -48 128 160 128 160 0
jset -15 0 8 0 8 0 4 0 0 0
jset -15 0 8 0 8 0 4 0 8.28 0
crack 48 56 64 56
crack 48 64 64 64
crack 48 96 64 96
crack 48 24 64 24
crack 64 24 64 57
crack 48 24 48 57
crack 64 64 64 97
crack 48 24 48 97
change mat=2 con=3
del area 2
save b81.sav
restart b81.sav
gen edge 20
bound str -20.6e6 6.2 -39.9e6
insitu str -20.6e6 6.2 -39.9e6 szz -37e6
grav 0 -10
prop mat=1 d=2700 k=33.4e9 g=19e9
prop mat=1 jkn=50e9 jks=5e9 jf=0.6
prop mat=1 kn=150e9 ks=5e9 jf=0.6
prop mat=5 d=2700 k=33.4e9 g=19e9
prop mat=5 jkn=50e9 jks=5e9 jf=0.4
prop mat=5 kn=50e9 ks=5e9 jf=0.4
change jcon=2 jmat=1
*cohesion(ore)= 31
prop mat=2 d=3500 k=33.4e9 g=19e9 coh=31e6
be gen -49.161 -1 129
be mat=5
be fix 0 -300 280 0
be stiff
change jmat=1
mscale on
damp auto
cyc 200
pr max
save b83.sav
reset hist
reset disp
change 48 64 64 96 con=0
hist unbal
hist n=50 sxx 56 59 ty 1
hist ydis 56 59
hist yvel 56 59
hist sxx 56 62
hist ydis 56 62
hist yvel 56 62
cyc 400
pr max
save b84.sav
prop mat=4 d=2300 k=.17e6 g=.056e6 coh=.75e6
change 48 64 64 96 mat=4 con=3
hist unbal
hist n=50 sxx 56 59 ty 1
hist ydis 56 59
hist yvel 56 59
hist sxx 56 62
hist ydis 56 62
hist yvel 56 62
cyc 400
pr max
save b84f.sav
del 48 64 24 56
hist unbal
hist n=50 sxx 56 59 ty 1
hist ydis 56 59
hist yvel 56 59
hist sxx 56 62
hist ydis 56 62
hist yvel 56 62
cyc 400
pr max
save b85f.sav
restart b85.sav
prop mat=4 d=2300 k=.17e6 g=.056e6 coh=.75e6
change 48 64 64 96 mat=4 con=3
hist unbal
hist n=50 sxx 56 59 ty 1
hist ydis 56 59
hist yvel 56 59
hist ydis 56 62
hist yvel 56 62
cyc 400
pr max
save b85f.sav
stop
LIST OF PUBLICATIONS FROM THIS RESEARCH

