DEVELOPMENT OF A KNOWLEDGE-BASED SYSTEM FOR OPEN STOPE MINE DESIGN

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ABSTRACT

Various open stope design methodologies have been developed previously, most involving empirical approaches, while more modern design methodologies have incorporated numerical stress analysis. To date, however, numerical analysis has had limited success in practical open stope design. This has been mainly due to the lack of formalised design methodologies incorporating numerical analysis, and the difficulties users have encountered in operating the numerical programs effectively. The research presented in this thesis has attempted to address these two independent topics within the context of open stope design, and has concentrated specifically on aspects of open stope design in narrow, steeply dipping, tabular orebodies.

In the first part of the thesis, numerical analysis methodologies adopted in other engineering disciplines are shown to be inapplicable for use in rock mechanics and new methodologies are developed specifically for open stope and pillar design, utilising boundary element stress analysis programs. A conceptual modelling approach is taken, whereby site characterisation data are used to select rock mass models, and to each of these is linked an appropriate numerical analysis program.

In the second part of the thesis, the difficulties encountered in implementing the developed methodologies in practice are addressed, and a knowledge-based systems approach is proposed as a means of incorporating open stope design methodologies in a program. A narrow task domain, that of crown pillar design, is selected for a prototype knowledge-based system. The developed system consists of two parts: an intelligent modelling tool for conceptualising a rock mass and then selecting the appropriate analysis technique, and intelligent front-ends to boundary element programs to guide users in their data input.

Finally, a case study of the No. 8 orebody at South Crofty mine, Cornwall, using the new open stope and pillar design methodologies and the crown pillar design knowledge-based system, is presented. The case study illustrates the advantages of utilising these methodologies and validates the basic premise that knowledge-based systems can significantly enhance an engineers ability to design open stope layouts. The key areas for further research have been established and are included.

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LIST OF SYMBOLS

Symbols	Definition
A	Factor A of stability number, N
AI	Artificial intelligence
B	Factor B of stability number, N
BF	Bolting factor
C	Factor C of stability number, N
C	Boundary
CAD	Computer aided design
cm	Centimetre
CSIR	South African Council for Scientific and Industrial Research
CSIRO	Commonwealth Scientific Industrial Research Organisation
DCF	Discounted cash flow
DOS	Disk operating system
DTH	Down-the-hole drill
E	Young's modulus
E	Evidence assertion
ES	Expert system
ϵ_3	Extensional strain
ϵ_c	Critical extensional strain
F	Factor of safety
$ft.$	Foot
G	Shear modulus
GPa	Gigapascal
H	Hypothesis assertion
H	Depth below surface
H	Height
H_p	Pillar height
H_s	Specimen height
IBM	International Business Machines

IKBS	Intelligent knowledge-based system
J_a	Joint alteration number
J_n	Joint set number
J_r	Joint roughness number
J_v	Volumetric joint count
J_w	Joint water reduction factor
k	Spring stiffness
k_{li}	Local stiffness of pillar i
km	Kilometre
K	Ratio of horizontal stress to vertical stress
KBES	Knowledge-based expert system
KBS	Knowledge-based system
kgf/cm^2	Kilogram force per square centimetre
kPa	Kilopascal
λ	Specimen stiffness
λ_i	Stiffness of pillar i
lbs	Pound (weight)
$lit./min.$	Litres per minute
LHD	Load-haul-dump machine
LN	Measure of necessity
LS	Measure of sufficiency
μ	Value of an element of a linguistic variable
m and s	The empirical parameters of the Hoek-Brown criterion
m_i	The intact value for m (when $s = 1$)
m	Metre
mm	Millimetre
mS	Millistrain
MPa	Megapascal
$MRMR$	Mining rock mass rating
N	Stability number
NGI	Norwegian Geotechnical Institute
ν	Poisson's ratio
$O(H)$	Prior odds on the hypothesis
$O(H\|E)$	Posterior odds on the hypothesis given the evidence assertion
p	Load

ϕ_b	Basic friction angle
ϕ_r	Residual friction angle
psi	Pounds per square inch
p_x or p_{xx}	Normal stress in Cartesian co-ordinates
p_{xy}	Shear stress in Cartesian co-ordinates
q_{xi}	Force in x direction at node i
Q	NGI rock mass quality index
Q'	Modified NGI rock mass quality index
R	Region
ROR	Rate of return
RMR	Rock mass rating
RMR'	Modified CSIR rock mass rating
RQD	Rock quality designation
σ_1, σ_2, σ_3	Principal stresses
σ_c	Unconfined compressive strength
σ_H, σ_h	Major and minor horizontal stress
σ_I	Induced compressive stress
σ_n	Normal stress
σ_o	Unconfined pillar strength
σ_p	Compressive strength of a pillar
σ_p	Average axial pillar stress
σ_s	Shear stress
σ_t	Tensile strength
σ_V	Vertical stress
σ_x or σ_{xx}	Normal stress components in Cartesian co-ordinates
σ_{xy}	Shear stress component in Cartesian co-ordinates
S	Hydraulic radius (ratio of area to perimeter)
S	Deformation
S_i	Spacing of joint set i
Sn	Tin
SRF	Stress reduction factor
τ	Shear stress
UCS	Uniaxial compressive strength
u_{xi}	Induced displacement in x direction at node i
VCR	Vertical crater retreat mining method

V_p Pillar volume	
V_s Specimen volume	
W	Width
W_o	Excavation width
W_p	Pillar width
W_s	Specimen width
x, y, z	Cartesian co-ordinates
Zn	Zinc

CHAPTER 1
INTRODUCTION

1.1 Purpose and Content of Thesis

Numerical modelling has become an integral part of engineering rock mechanics design, as a means of predicting the response of a rock mass to engineering activity. Good excavation design requires a detailed knowledge of the material properties of the rock (including the rock structure) and the boundary loading conditions, if the excavation is to fulfil its desired role. Correct numerical analysis of the excavation should then be able to model the actual rock mass behaviour. Figure 1.1 schematically shows the interactions in rock mechanics between material properties, loading conditions and design problems.

There are a multitude of numerical modelling programs available, all with their own individual strengths and weaknesses. Numerical programs have not to date been widely used in mine design, but are increasingly being incorporated into mine design methodologies. There are many reasons why numerical programs have not been used more routinely, but the main problems have been the lack of formalised design methodologies incorporating numerical methods, and the difficulties users have encountered in operating the programs. This research has attempted to address these two independent topics, within the context of open stope design, and has concentrated specifically on aspects of stope design in narrow, steeply dipping, tabular orebodies. For this thesis, these are taken to be orebodies which are less than 5 m in width, and which dip at an angle greater than 50° to the horizontal.

In the first part of this thesis, the role and development of numerical modelling in mine design are examined, and programs and methodologies that are suitable for the specific design problem are selected. From these, clear new methodologies are developed for use by mining engineers. In the second part of this thesis, the difficulties encountered in implementing these methodologies in practice are addressed, and the new technology of knowledge-based systems is identified as being an innovative means of incorporating open stope design methodologies in

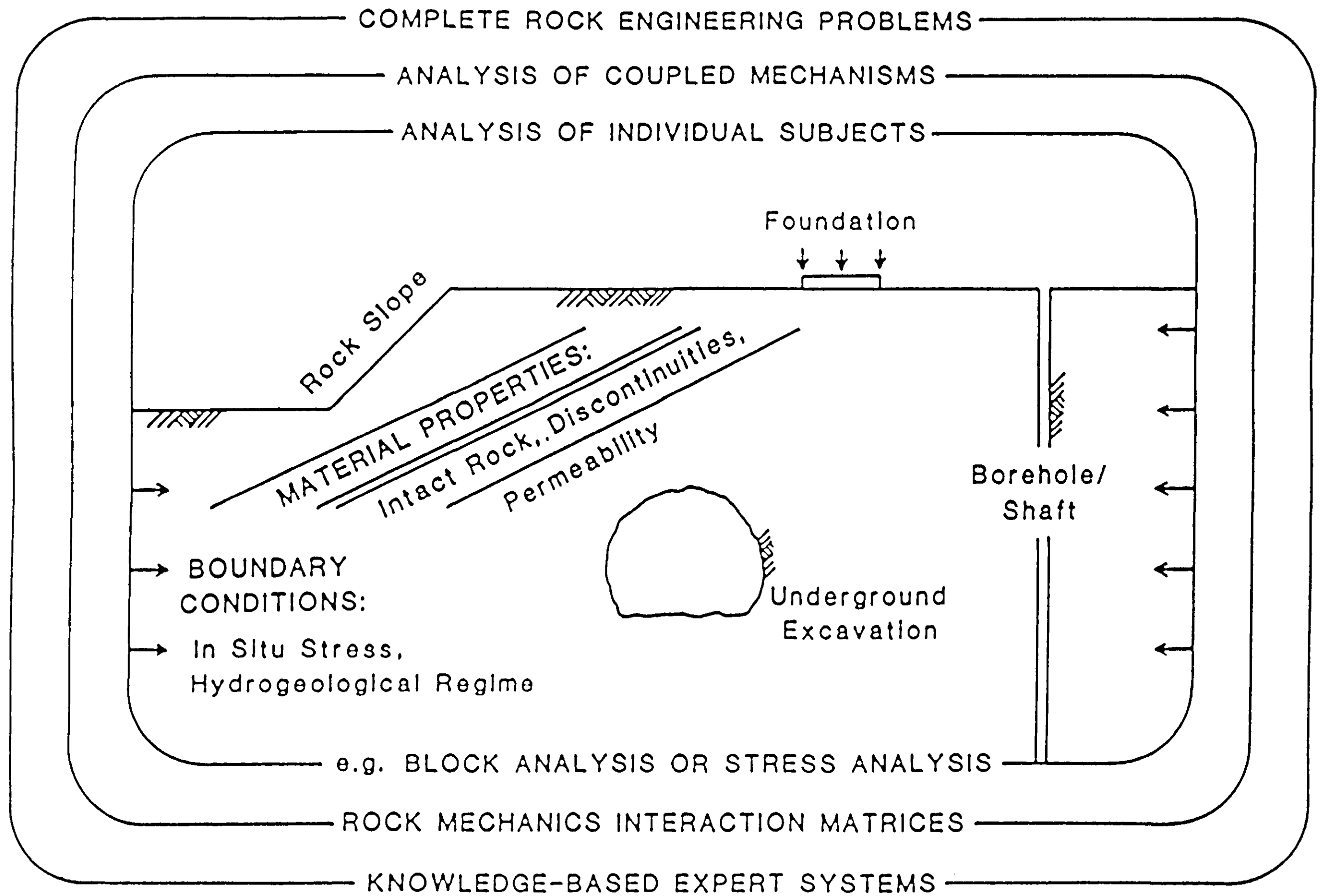


Figure 1.1 The interrelations of problems in engineering rock mechanics (after Hudson, 1989).

computer programs. A knowledge-based system for crown pillar design is developed to provide a tool for mining engineers, that allows pillar design to become a routine task. Figure 1.2 gives a flowchart of the thesis structure.

The rest of this chapter is devoted to exploring the role of rock mechanics in open stope mine design, detailing common problems encountered, and concludes with a discussion on the development of fully computerised mine design programs. The aim of such systems is shown not to be a means of replacing engineers, but as a way of freeing them from laborious, repetitive tasks, allowing them to concentrate on more creative tasks. By allowing engineers to focus on the design problem itself, rather than solely on the mechanics of solving a problem, the result ultimately should be better design. Finally, a detailed structure of the thesis is given in Section 1.4.

1.2 Rock Mechanics In Open Stope Mine Design

1.2.1 Introduction

The basic objective in the design of an underground mine structure is to maximise profits, whilst at the same time ensuring safe and efficient extraction of a high proportion of the *in situ* ore reserve. Rock mechanics input to mine design is aimed at maximising the economic potential of an orebody by predicting and controlling rock mass displacements.

Logically, rock mechanics should form the backbone of any mining operation. However, within the mining industry it is seen as a specialised discipline (Ferguson, 1985a). Rock mechanics should not be thought of as a narrow field, and in a sense may be regarded as the basic mining science (Brady and Brown, 1985). One of the main reasons why the mining industry has not fully appreciated rock mechanics is the fact that rock mechanics has been viewed as a discipline that cannot be applied in a practical and cost effective way (Bawden *et al.*, 1988a). With the increased scale of operations at modern mines, in order to maximise profitability, coupled with the increased depth of mining and the requirement for greater mine safety, the need for rock mechanics to play a central role in mine design is becoming more apparent. The next section discusses the positive economic impact that rock mechanics design can have on open stope mining operations.

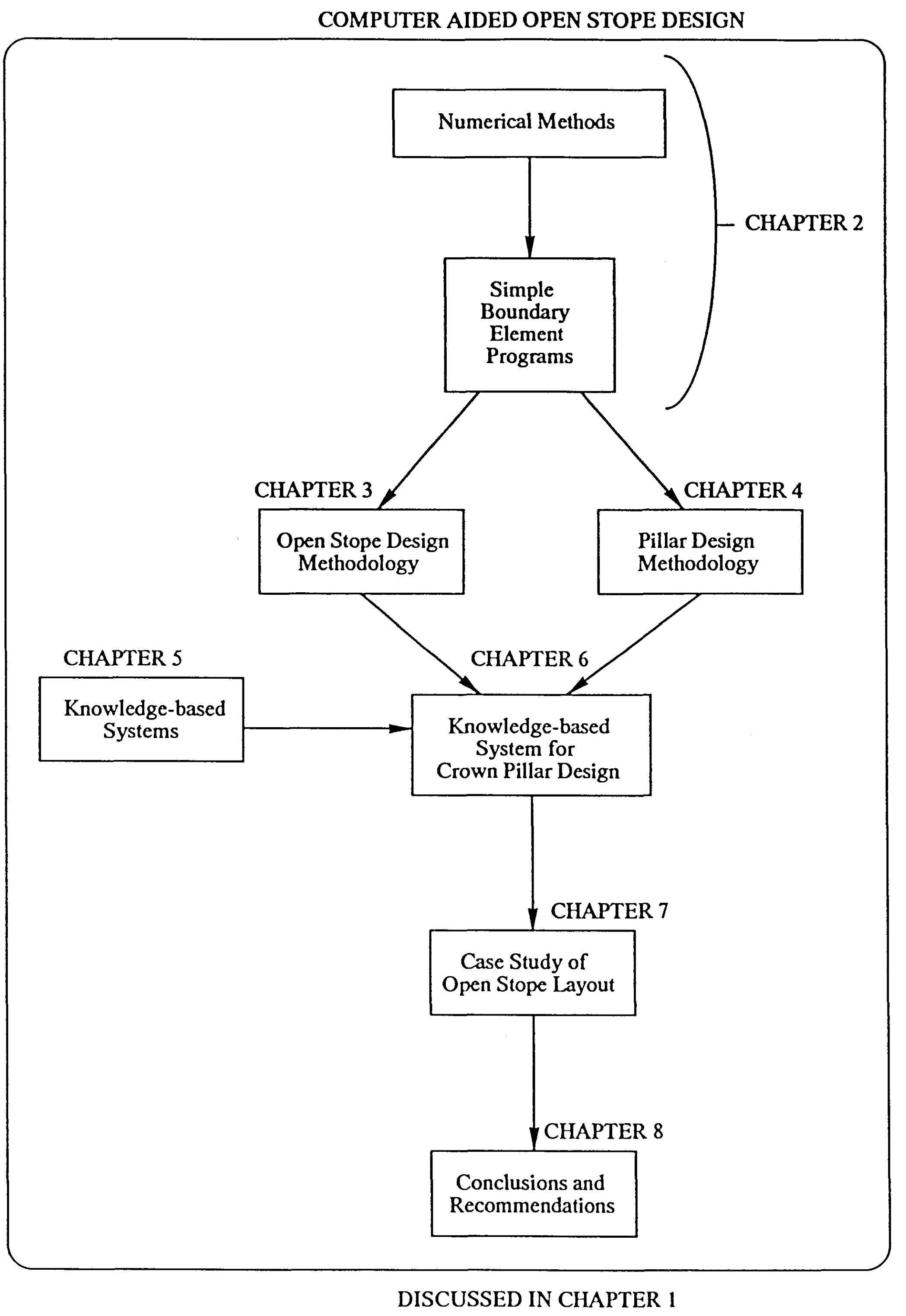


Figure 1.2 Flowchart of thesis structure.

1.2.2 Design Problems in Open Stoping

Sound rock mechanics design of stopes, pillars, ground support and blasting can have a major impact on the profitability of any mining operation. Mining profits come from the ore extracted from the stopes and so profitability is directly related to stope stability. Grade dilution is one of the most serious potential economic consequence resulting from improper geometric design. Open stoping, by its nature, does not allow flexibility in following irregular ore-waste contacts, and so incurs a penalty of either leaving ore behind, or mining waste with the ore, so reducing the mined grade. The move to large stopes, with little ability to apply systematic support to stope surfaces can also increase problems with dilution. Table 1.1 shows a simple discounted cash flow (DCF) rate of return (ROR) calculation for a fictitious orebody. For dilution of the order of 40%, a negative rate of return occurs.

Open stoping, in common with the other naturally supported mining methods, commits ore to pillar support, to control near-field displacements, as well as to ensure global stability of the mine structure. Design of pillars to minimise the amount of ore committed to pillar support is seen by the mining industry as being of primary importance. Generally, pillars are recovered subsequently, but their condition prior to recovery determines whether this is a simple inexpensive task, or not. Good design should attempt to preserve the mineable condition of unmined ore.

The ability to predict and control rock mass performance has also been shown to lead to a lowering of mining costs in areas not directly related to rock mechanics. Merrill and Yardley (1982) noted that cost reductions in areas such as drilling, blasting, scaling, mucking and transportation could be achieved.

1.2.3 Modelling in Open Stope Design

Mine stability analysis is rarely undertaken explicitly in mine design. This is because the zone of influence of stopes (the region where the effect of the stopes on the pre-mining stress field is significant) is large relative to virtually all other mining excavations. Stope design therefore exercises a dominant role in the location, design and operational performance of other excavations which sustain mining activity (Brady and Brown, 1985).

Dilution (%)	0%	10%	20%	30%	40%
Tonnage	2,500,000	2,780,000	3,130,000	3,570,000	4,170,000
Grade	20.0%	18.0%	16.0%	14.0%	12.0%
Mining Rate	360,000	360,000	360,000	360,000	360,000
Mining Life, yrs.	6.9	7.7	8.7	9.9	11.5
Metal recovery	85%	85%	85%	85%	85%
Zn / short ton (*lbs*)	340	304	272	238	204
Zn / year (*lbs*)	1.22×10^8	1.09×10^8	0.98×10^8	0.85×10^8	0.73×10^8
Mine Revenues/yr. (25*c*/*lb*)	\$30.50 $\times 10^6$	\$27.25 $\times 10^6$	\$24.50 $\times 10^6$	\$21.25 $\times 10^6$	\$18.36 $\times 10^6$
All Mine Costs (\$40/ton)	\$14.4 $\times 10^6$	\$14.4 $\times 10^6$	\$14.4 $\times 10^6$	\$14.4 $\times 10^6$	\$14.4 $\times 10^6$
Operating profits	\$16.10 $\times 10^6$	\$12.85 $\times 10^6$	\$10.10 $\times 10^6$	\$6.85 $\times 10^6$	\$3.9 $\times 10^6$
Taxes (50% of O.P.)	\$8.05 $\times 10^6$	\$6.43 $\times 10^6$	\$5.05 $\times 10^6$	\$3.43 $\times 10^6$	\$1.98 $\times 10^6$
Net Profit	\$8.05 $\times 10^6$	\$6.43 $\times 10^6$	\$5.05 $\times 10^6$	\$3.43 $\times 10^6$	\$1.98 $\times 10^6$
Capital Cost of Mine	\$2.5 $\times 10^6$	\$2.5 $\times 10^6$	\$2.5 $\times 10^6$	\$2.5 $\times 10^6$	\$2.5 $\times 10^6$
DCF ROR	25.5%	18.9%	13.5%	6.0%	-1.5%

Table 1.1 Example of importance of dilution on discounted cash flow rate of return (after Bawden, 1988a).

The effective implementation of rock mechanics in stope design has occurred only in the last 20 years. Prior to this, design was mainly by precedent practice with little regard to the mechanics of the problem. Rock mass classification schemes (Bieniawski, 1976; Barton *et al.*, 1974) attempted to characterise rock masses and link design procedures directly. These schemes were useful for characterising a rock mass, but the case histories on which they were based were for shallow excavations, and their design procedures were for tunnels, rather than mining excavations. Laubscher (1977, 1984) modified Bieniawski's scheme for mining applications but the mechanics of the problem were still not accounted for properly. The effect of stress change, due to mining, was determined using an adjustment factor which was quite unsuitable. Rock mass classification schemes in general tend to perpetuate existing practice and do not highlight work that is conservative.

In recent years numerical modelling has been used with success in all areas of stope design – stope and pillar location and dimensioning, detailed analysis of potential instability, and stope sequencing (Stephansson, 1985; Pariseau *et al.*, 1985; Bywater *et al.*, 1983). However, in many cases the full potential of numerical analysis has not been realised, because the valid application of these programs was not appreciated. As Brady and St. John (1982) noted, criteria have to be defined which qualify particular modelling procedures to specific design problems.

Another major problem encountered in the use of numerical models is the selection of input parameters. This includes the *in situ* strength and deformation properties of all rock types present, the location, attitude and mechanical properties of major persistent structural features, and the magnitudes and orientations of the *in situ* stresses. The numerical programs tend to be large and complicated, and require skill to use them effectively and efficiently. For the above reasons, numerical modelling to date has largely been conducted as a one-run exercise by consultants. Modelling, as Starfield and Cundall (1988) noted, should never be perceived as a one-run exercise, but should involve many runs, to determine the sensitivity of the results to input parameters and assumptions made.

Simple conceptual models of rock masses are a means of representing a complex rock mass in a form that contains the basic features of the rock mass (Starfield and Detournay, 1981; Brady, 1987; Pan, 1989). Using conceptual models, it is possible to classify rock masses in terms of their mechanical behaviour – continuum, pseudo-continuum, or discontinuum (Figure 1.3). Once the rock mass has been classified, it is possible to link computational schemes that are appropriate to the rock mass and to the design problem. Using this methodology good open stope design can be undertaken by geotechnical engineers at a mine site.

1.3 Computer Aided Open Stope Design

1.3.1 Introduction

Computers are widely used in the planning and design stages of mining operations, but there has been little standardisation of hardware or software, and most tasks are covered by individual programs. Computers have been used successfully for designing open pit mine envelopes, and calculating the associated ore and waste contents, since the early 1960's (Wilke *et al.*, 1984). However, there have been few general programs for the modelling of underground mining, and most programs have concentrated on one aspect of the design process *e.g.* short term planning of sublevel stopes (Muge and Pereira, 1979).

1.3.2 Proposed Computer Aided Mine Design Systems

Recently, various proposals have been presented for integrated underground mine design systems for use in feasibility studies, financial appraisals and day-to-day geological and mine planning work (Arcamone *et al.*, 1984; Ferguson, 1985b; Brown *et al.*, 1988). These proposed systems are all similar in concept, having various modules that can analyse a separate aspect of a problem and then pass results on to a central database, where other modules can access this information. Ferguson (1985b) proposed a system comprising of nine modules covering all aspects of the design process (Figure 1.4). Each module contained algorithms and a database. The algorithms would be for solving design calculations, optimisations of systems and cost calculations. The database would combine manufacturers data and precedent experience from previous projects, existing mines and technical journals. Work on a specific project would require a project database with a parallel structure to the mining design system (Figure 1.5). Each activity would have two

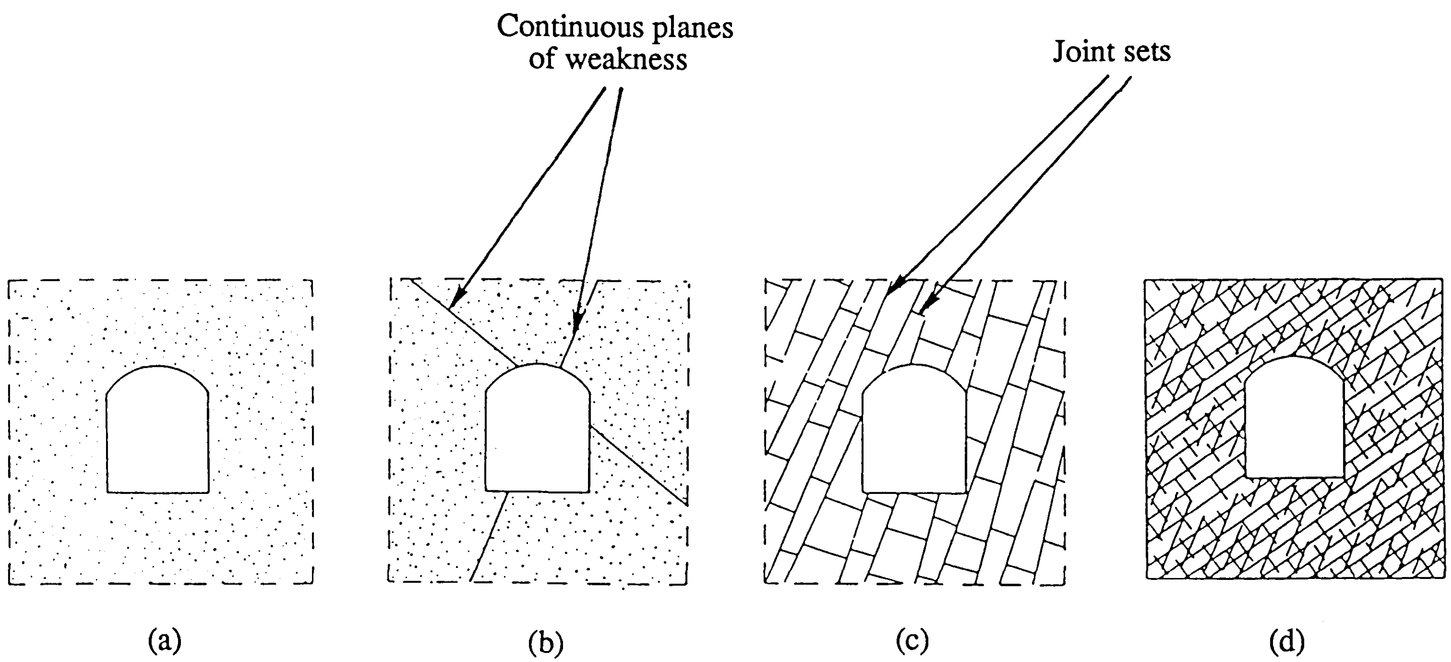


Figure 1.3 Conceptual models relating rock structure and rock response to excavation (after Brady, 1987).

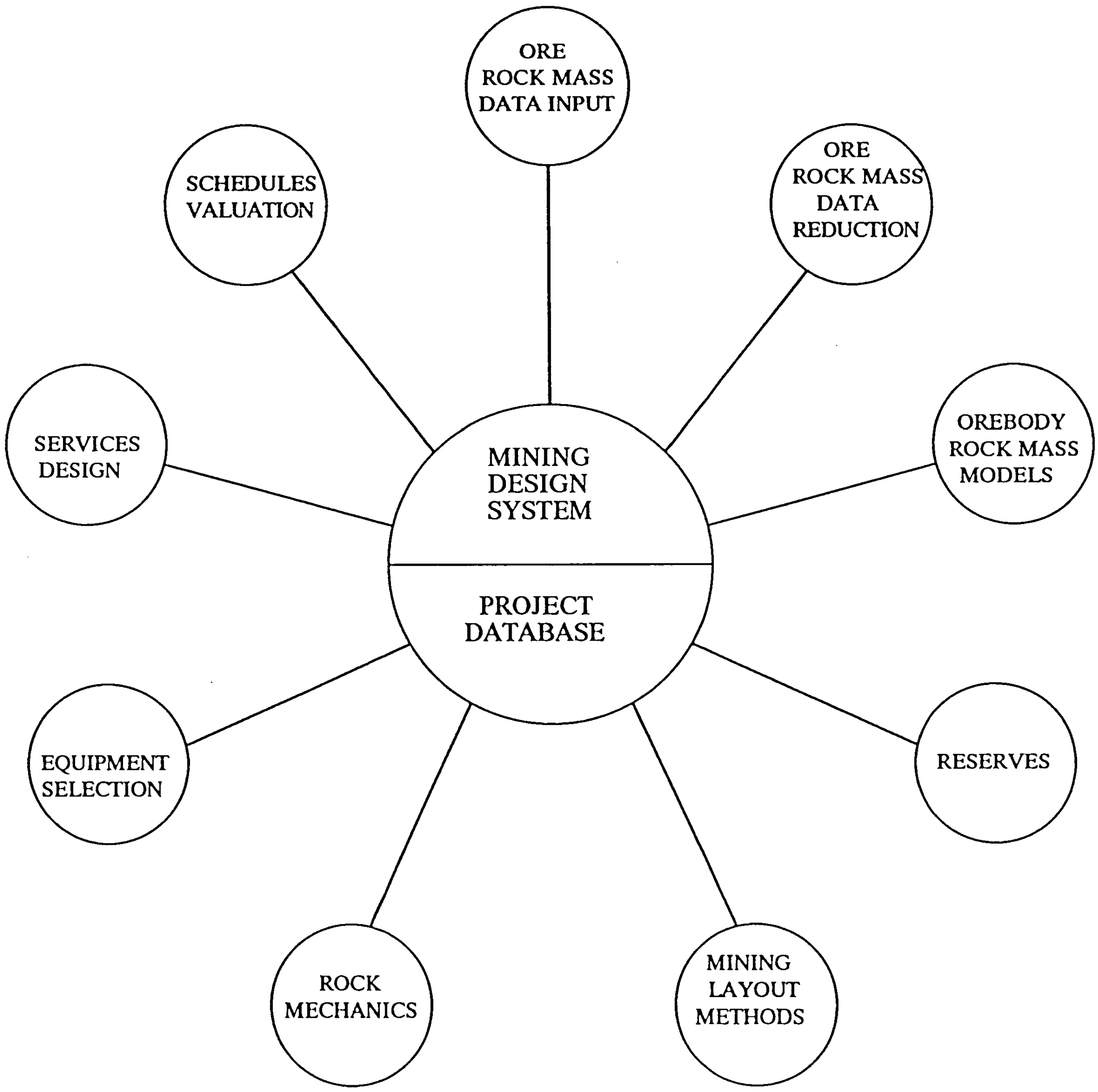


Figure 1.4 Proposed computer-based mining design system (after Ferguson, 1985b).

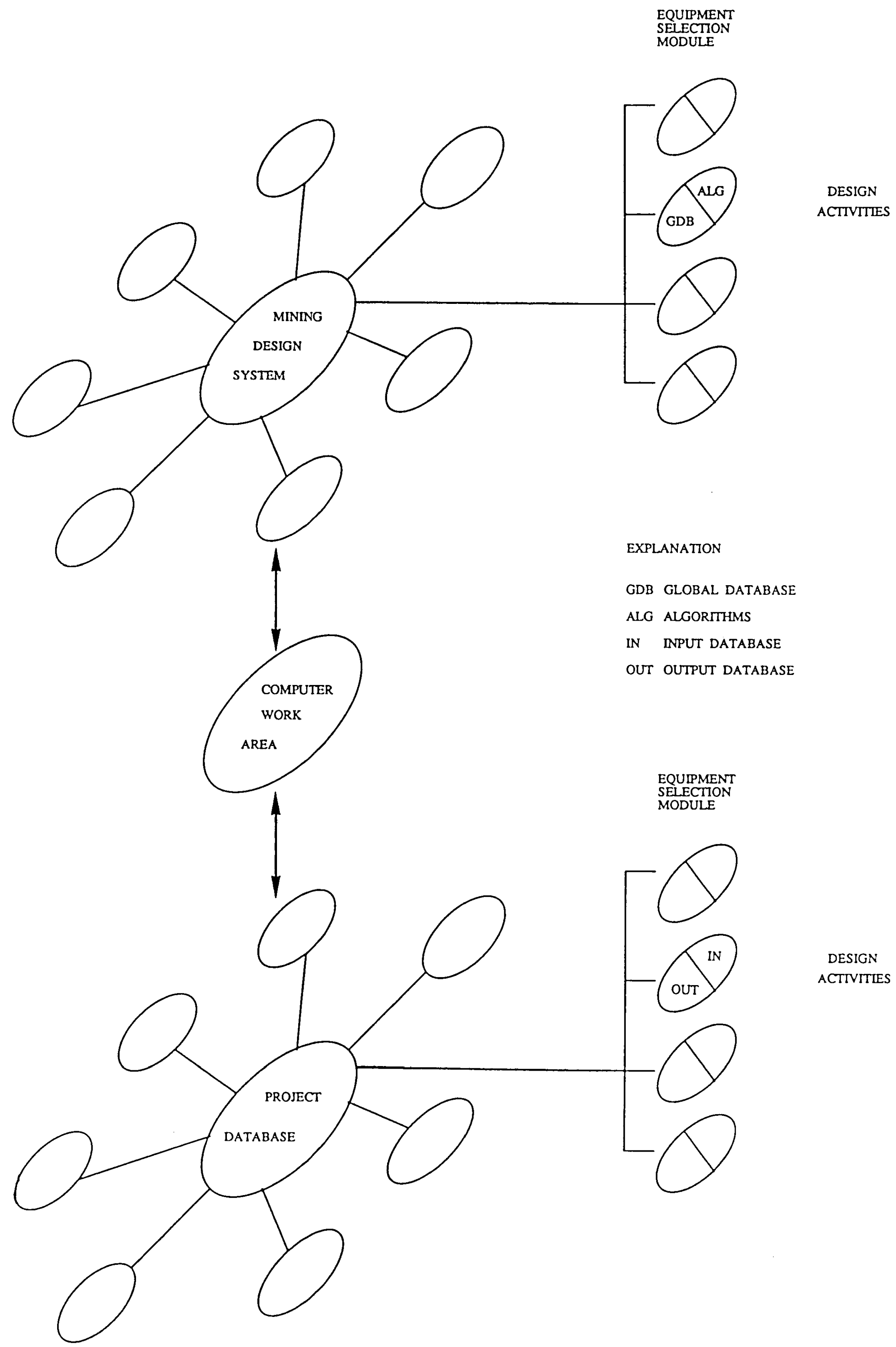


Figure 1.5 Relation between mining design system and databases (after Ferguson, 1985b).

databases within each project module – one being the input database for project specific information from relevant associated modules, the other being an output database made up of information files, enabling the transfer of design and cost data to the relevant modules.

The aim of such a system is to free the engineer from much of the routine elements of mine planning, such as manipulation and modification of large data sets; repetitive calculations, both simple and complex; numerous geological and engineering drawings; tabulation and scheduling of physical quantities. Much of this work is of a repetitive nature, time consuming, and prone to errors. The envisaged system would allow engineers to interact and try out their ideas. The system is not intended to replace engineers, but to allow them to spend more time on the intellectual elements of mine planning, which should ultimately lead to improved design.

The structure of these proposed systems is essentially that of a large knowledge-based system with access to available engineering software and expert knowledge. Brown (1986) described a research proposal to produce an integrated mine design program, incorporating computer aided design (CAD) as well as elements of knowledge-based systems. The envisaged system was proposed to be a series of computer packages interconnected by a logical sequence of decisions that an engineer would make when designing a mining system. The research undertaken by this author has formed part of this proposal, concentrating specifically on the stress controlled failure submodule of the Rock Mechanics master module (Figure 1.6). This submodule should contain proven two and three-dimensional stress analysis programs, incorporating interactive data input, analysis, and display of results in tabular or graphic format.

1.4 Structure of Thesis

In Chapter 2, the available numerical methods in engineering rock mechanics are investigated, in order to determine their applicability for open stope design. The underlying principles, strengths and weaknesses, and application in mine design are discussed. The boundary element method is identified as the most suitable method for open stope design, and a suite of boundary element programs suitable for open stope and pillar design are evaluated.

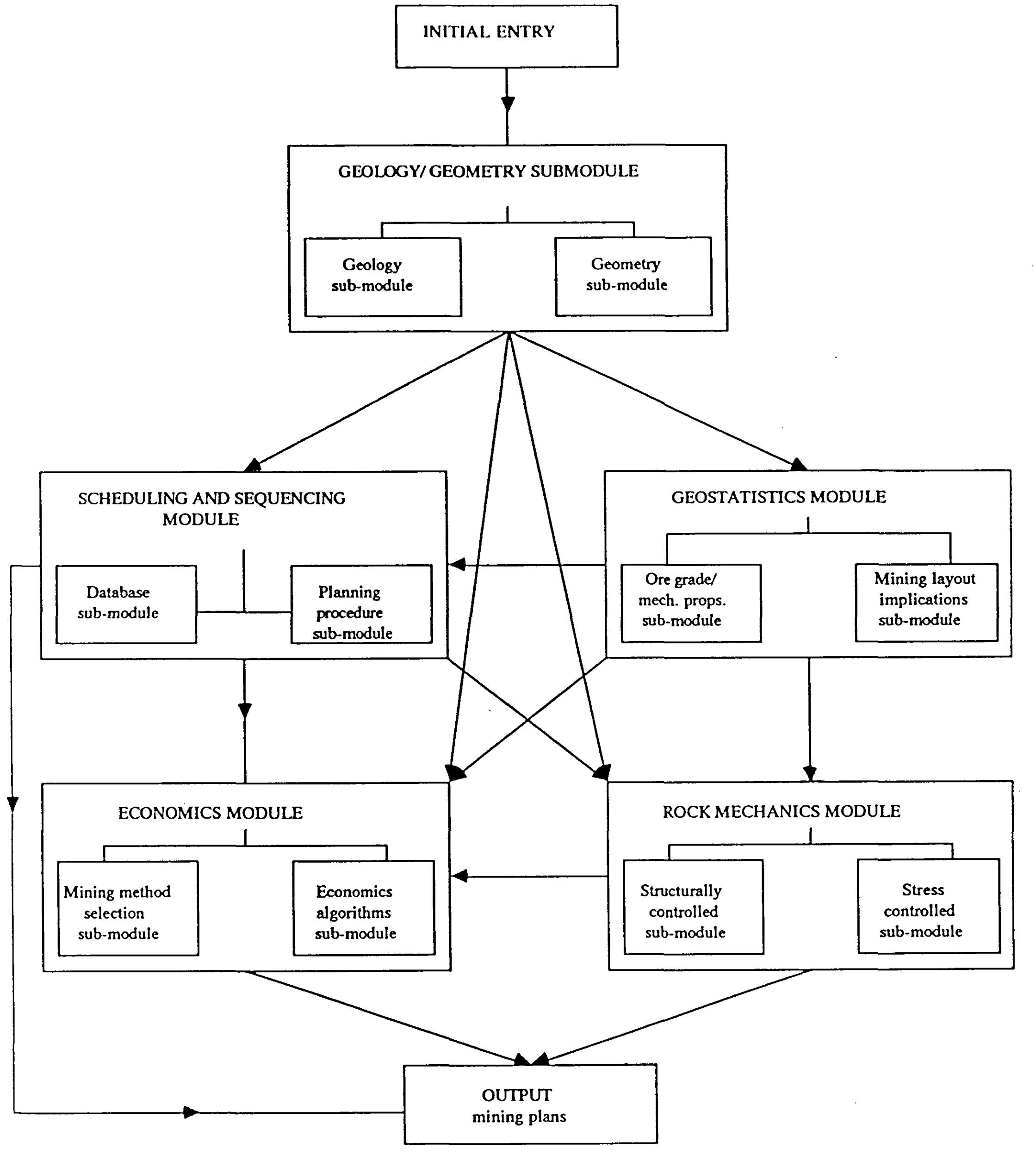


Figure 1.6 Proposed knowledge-based system for underground metalliferous mine design (after Brown, 1986).

Chapter 3 describes the open stoping mining method and shows that it has become one of the most popular and versatile mining methods for use in underground metalliferous mines. A discussion on rock mass modelling results in the adoption of a numerical modelling approach that uses conceptual models, and which is specifically designed for use in engineering rock mechanics. There follows a critical review of the various approaches to open stope design, leading to the development of an open stope design methodology, using numerical analysis, that is firmly based on the observational principle.

Chapter 4 examines the pillar failure mechanisms that have been reported in open stope pillars, in an attempt to determine the various modes of failure. There follows a critical review of the various approaches to open stope pillar design. Finally, a pillar design methodology is proposed that utilises numerical analysis as a means of determining the stress distribution within pillars, and a failure criterion for determining the strength and behaviour of pillars.

Chapter 5 introduces knowledge-based systems and describes what they are and how they differ from conventional programs. A methodology for the development of knowledge-based systems is discussed, covering all aspects from task selection, through knowledge acquisition, to system implementation. Finally, a review of various modes of use of knowledge-based systems, concentrating on geotechnical applications, highlights their potential use in mine design. The development of intelligent front-ends to existing numerical programs, and intelligent modelling tools are shown to be areas where this new technology can be used in engineering rock mechanics design.

Chapter 6 brings together the numerical analysis programs of Chapter 2, the design methodologies of Chapters 3 and 4, and the technology of knowledge-based systems described in Chapter 5. Simple conceptual models are developed from site characterisation data allowing, for specific design problems, the appropriate design methodology and numerical program to be selected. A simple knowledge-based system for the design of crown pillars in narrow, steeply dipping, tabular orebodies is developed, using a knowledge-based system development program, Insight 2+. This problem area was chosen, as crown pillar optimisation is seen to be of primary importance by many mines.

Chapter 7 demonstrates the use of the developed methodologies for open stope and pillar design, described in Chapters 3 and 4, respectively. The No. 8 orebody at South Crofty mine, Cornwall, U.K. was selected for a case study, and various pillar and stope layouts were modelled. Recommendations for monitoring to allow calibration of the numerical models are given, together with advice for future stope design.

The conclusions reached and contributions made by the research are given in Chapter 8, together with suggestions for further research.

CHAPTER 2
NUMERICAL METHODS FOR OPEN STOPE DESIGN

2.1 Introduction

Chapter 1 discussed the common requirement in mining excavation design, of determining the stress and displacement distribution in the rock mass surrounding an excavation. A particular need is for methods which allow parameter studies to be undertaken quickly, so that a number of possible mining layouts can be evaluated, to determine which is the most suitable, from a geomechanical viewpoint. The earliest attempts at modelling mine structures used physical models. Their main aim was to identify flaws in the design which might lead to extensive failure. However, physical modelling was time consuming and expensive, and it was difficult to maintain similitude in the material properties. For mine structures that could be modelled as two-dimensional, base friction modelling (Bray and Goodman, 1981) provided a useful and inexpensive method of design evaluation. It was especially useful where discontinuities had a dominant role in the rock mass response.

These physical models, however, yielded little or no information about the stresses and displacements within a rock mass. Photo-elastic methods were developed which allowed contour plots of the principal stresses, within the sample medium, to be determined. As with the physical models, the photo-elastic method suffered from the problem that it was a costly and time consuming way of predicting the performance of an excavation.

Analytical solutions provided a means of determining stresses and displacements around excavations without the need for a physical model. There exists various closed-form solutions for the elastic stress distribution around simple shaped openings, *e.g.*, circular or elliptical openings (Brady and Brown, 1985). These allowed an insight into the stress distributions around more complex excavation shapes, but for detailed work around complex excavation shapes they were of limited use.

The development of computational procedures, in the last 20 years, that allow the modelling of stresses and displacements around excavations of any shape, in a variety of mediums, has resulted in numerical modelling superseding all other modelling techniques. The computational schemes have enabled the design and analysis of mining layouts to be carried out on a routine basis, and are now firmly established in engineering rock mechanics. Computational methods of stress analysis for boundary value problems can be divided into two distinct classes: differential methods and integral methods. Differential methods require that approximations are made throughout the problem domain, whilst integral methods require approximations to be made only on the boundary. *Finite difference* and *finite element* techniques fall within the class of differential continuum mechanics. *Boundary element* techniques belong to the integral continuum mechanics class. Discontinuum mechanics, whilst falling within the broad class of differential methods, will be considered separately from differential continuum methods. A recent development has been the combining of differential and integral methods to produce *hybrid* or *coupled* method programs.

This chapter briefly reviews the available numerical techniques for stress analysis around open stopes. Details on the theory and formulations of the various methods are not discussed, but the key references on the methods are given. Assessments about the applications and limitations of the methods for open stope and pillar design are discussed. The problem to be solved is one of initial design, that requires a quick, efficient tool, but which can take into account the mechanics of failure. To this end, the boundary element method is highlighted as being the most appropriate numerical modelling technique for open stope design in deep, hard rock mines, and attention is focused on its development and application.

A suite of boundary element programs were adapted for use on a microcomputer and their individual applications and limitations are discussed in Section 2.3.5, with reference to the design of open stopes in narrow, steeply dipping, tabular orebodies. These boundary element programs are subsequently used in the developed design methodologies for open stopes and pillars that are described in Sections 3.5 and 4.4, respectively. In Chapter 6 a knowledge-based system is developed that enables mining engineers to use the most appropriate numerical analysis program in the design of crown pillars, in a correct and effective manner. Finally, in Chapter 7, the adapted boundary element programs are utilised in a case study of the design of an open stope layout at South Crofty mine, Cornwall.

2.2 Finite Element Method

2.2.1 Methodology

Differential continuum methods may be used to solve boundary value problems in elasticity and plasticity. There are essentially two methods in the group of differential continuum methods – the finite difference and finite element methods. Finite difference methods are rarely used in the solution of problems in statics (Brown, 1987) and so shall not be considered here. The finite element method is a general numerical technique for the solution of partial differential equation systems, subject to appropriate boundary conditions and initial conditions (Zienkiewicz, 1977; Hinton and Owen, 1979).

In the finite element method the continuum is divided into discrete non-overlapping regions, known as elements, which provide a physical approximation to the continuity of displacements and stresses within the continuum. Figure 2.1 (a) shows a cross section of an underground excavation in an infinite body subject to initial stresses p_{xx}, p_{yy}, p_{xy}. Figure 2.1 (b) shows the domain divided into a set of elements. A representative element is illustrated in Figure 2.1 (c), with the points i, j, k defining the nodes of the element. The problem is to determine the state of total stress, and the excavation-induced displacements throughout the assembly of elements. The governing partial differential equations are solved at the nodes, where adjacent elements are connected. The system of equations is large but sparse, *i.e.*, although there are a large number of unknown parameters, and hence a correspondingly large number of equations, each equation contains only a few of the unknown parameters explicitly. The values of stress and strain at or between the nodes are then approximated using interpolation functions. Zienkiewicz (1977) gives full details about the development of the method, and its use in a wide range of engineering applications.

2.2.2 Application of Finite Element Method

Finite element methods have been used widely in open stope and pillar design. Recently, there has been a trend towards the use of boundary element methods, but many mines still use finite element analysis, especially if they wish to explicitly model rocks with different material properties, together with non-linear or elasto-plastic material behaviour. The development of joint elements (Goodman and St. John, 1977) has allowed the non-linear stiffness and shear strength

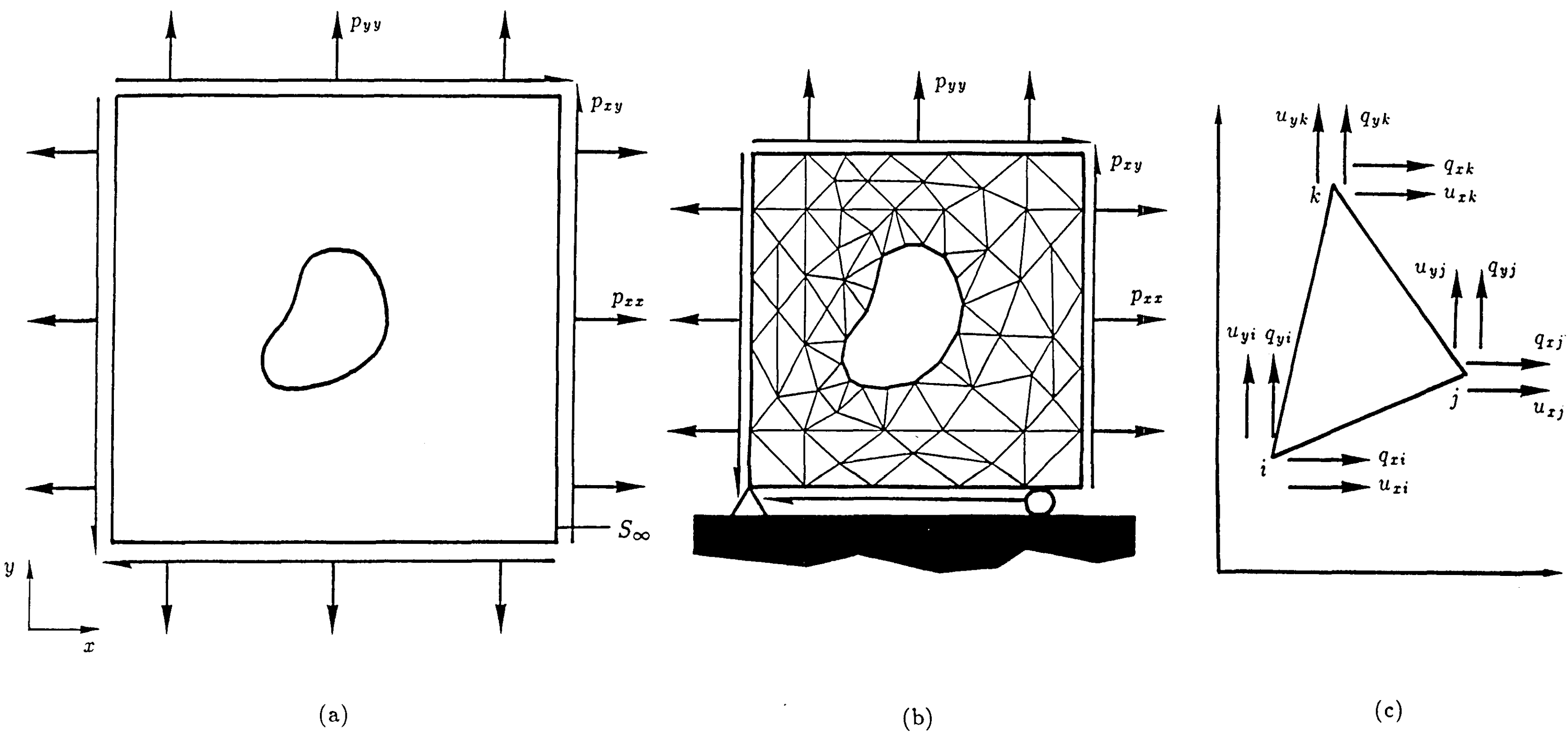


Figure 2.1 Development of a finite element model: (a) the continuum problem; (b) specification of element mesh and loading; (c) a representative triangular element (after Brady and Brown, 1985).

characteristics of discontinuities to be incorporated into finite element programs.

Meek and Kirby (1976) described the use of a simple elasto-plastic finite element program to model the stability of cemented fill in the copper orebody at Mount Isa mine, Australia. The mining method involved open stoping followed by recovery of pillars between the primary stopes. The object of the analysis was to determine the safe unsupported height of fill, and the height to which the fill had to be cemented. Pariseau *et al.* (1985) described the use of an elasto-plastic finite element program for modelling vertical crater retreat stopes at two mines. They compared predicted displacements and measured displacements to calibrate their program to the mine site. They found that despite the apparent difficulty of specifying a large number of independent rock mass properties, two scale factors – one for the elastic moduli and one for the strengths – were sufficient to calibrate the model. The calibrated model was then used to analyse alternative stope layouts and was shown to result in considerable savings in design studies, compared to full scale mine trials.

Pariseau (1975) and Agapito and Hardy (1982) used finite element methods in pillar design in order to try to model the development of failure zones within pillars. Stephansson (1985) described the use of finite element analysis at three mines in Scandinavia, for crown pillar design. The aim of all these studies was to optimise the size of the crown pillars (Figure 2.2).

2.2.3 Conclusions

Finite element methods have advantages over boundary element methods for modelling the development of failure zones around stopes and in pillars, as they are more able to model the complex rock behaviour during excavation, and the resultant stress redistribution. However, such analyses require large and complex finite element meshes, and for large numbers of excavations, use of the finite element method is usually prohibitively expensive, on a day to day basis.

The finite element method also suffers from the limitation that the outer boundary of the problem domain has to be arbitrarily defined. This can introduce inaccuracies into the solution because the far-field stress conditions may not be satisfied completely. This difficulty has been overcome by the development of infinite elements (Beer and Meek, 1981), and the development of hybrid, or coupled, finite element - boundary element programs (Section 2.5.2).

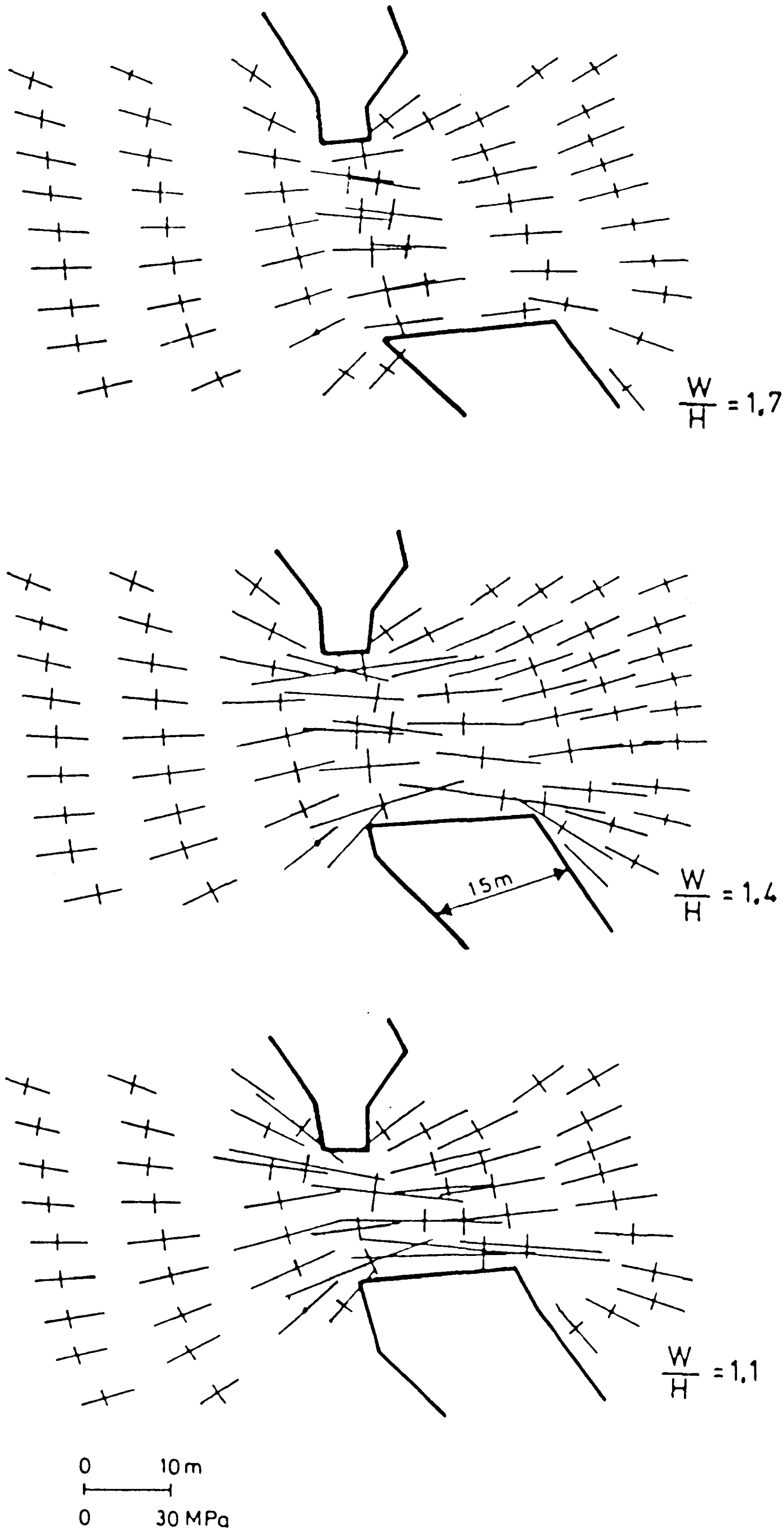


Figure 2.2 Finite element analysis of sill pillar No. 1 in the Green orebody, Rautuvaara mine, Finland, for various width to height ratios (after Stephansson, 1985).

2.3 Boundary Element Method

2.3.1 Methodology

In boundary element methods only the boundary, C, (Figure 2.3) is divided into elements. The numerical solutions use analytical solutions for simple singular problems in such a way as to satisfy, approximately, for each element, the boundary conditions in terms of imposed tractions and displacements. Since each of the singular solutions satisfies the governing partial differential equations in the region R (Figure 2.3), there is no need to divide R itself into a network of elements. Thus, the system of equations to be solved is much smaller than the system needed to solve the same boundary value problem using the finite element method. However, the equations are no longer sparse, but are fully populated. This effective unit reduction in the order of the problem gives integral methods significant advantages in computational efficiency over differential methods. This makes integral methods especially useful for solving three-dimensional problems, which are commonly met in mining rock mechanics (Watson and Cowling, 1985).

There are basically two different versions of the boundary element method; the *indirect* and the *direct* formulations. In the indirect formulations the physical problem is replaced by an equivalent problem for which the solution will be the same, providing that all the boundary conditions are the same.

The equivalent problem is that of an imaginary surface, corresponding to the original excavation boundary (Figure 2.4). The surface is divided into segments along which both normal and shear forces are applied. The stresses resulting from such forces applied in an infinite elastic region are known from the theory of elasticity (Timoshenko and Goodier, 1970). Influence functions that describe the effect that a force on any segment has on all the other segments along the boundary can be derived. These influence functions can then be used to derive a set of equations that must satisfy the boundary conditions for the problem. When solved, these equations give the force distribution on the segments that results in exactly the same stress distribution as for the actual physical problem. The calculated forces have no physical significance, so the method has become known as the *fictitious force* method.

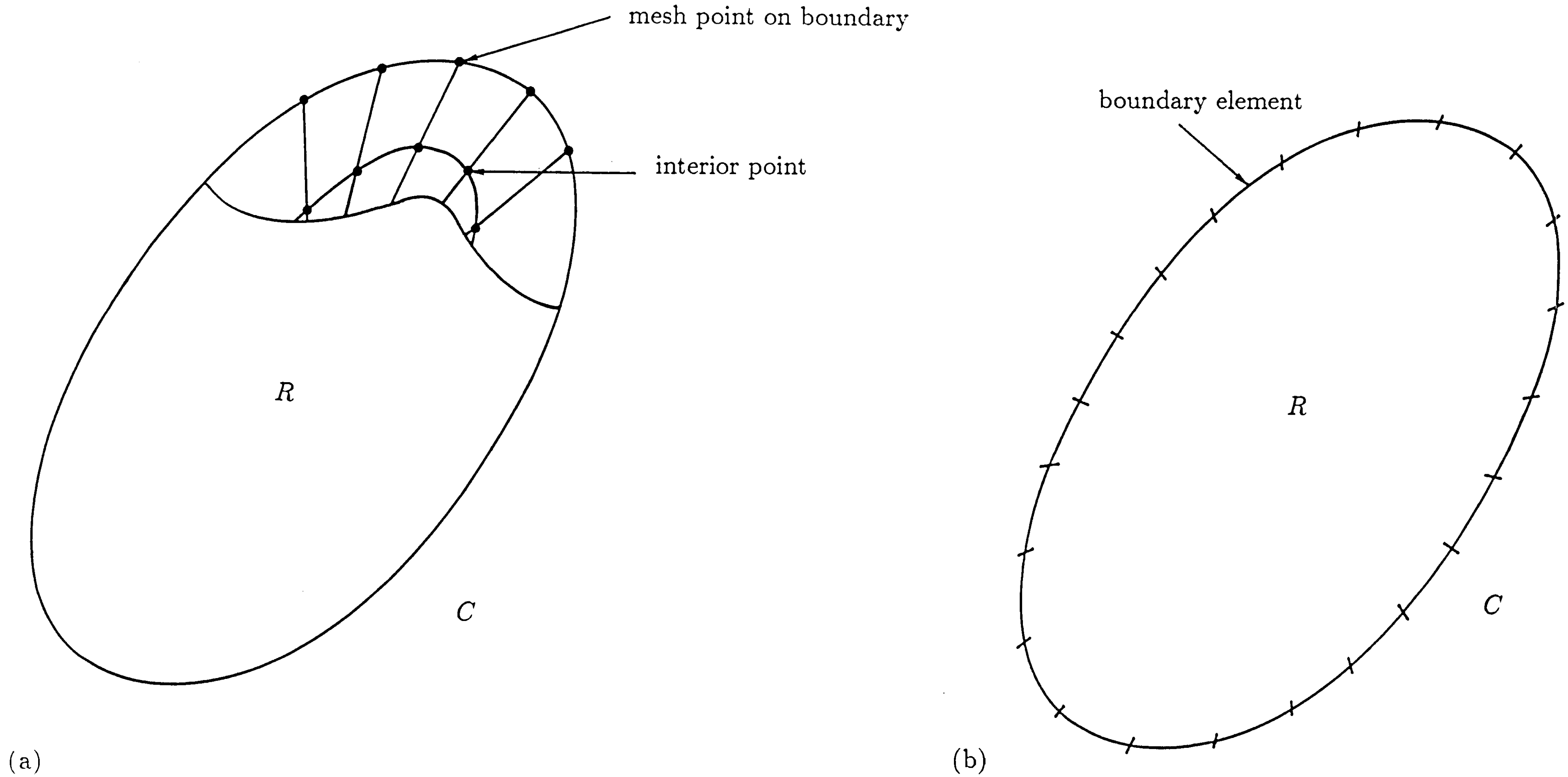


Figure 2.3 Finite element and boundary element idealisations: (a) finite elements; (b) boundary elements (after Crouch and Starfield, 1983).

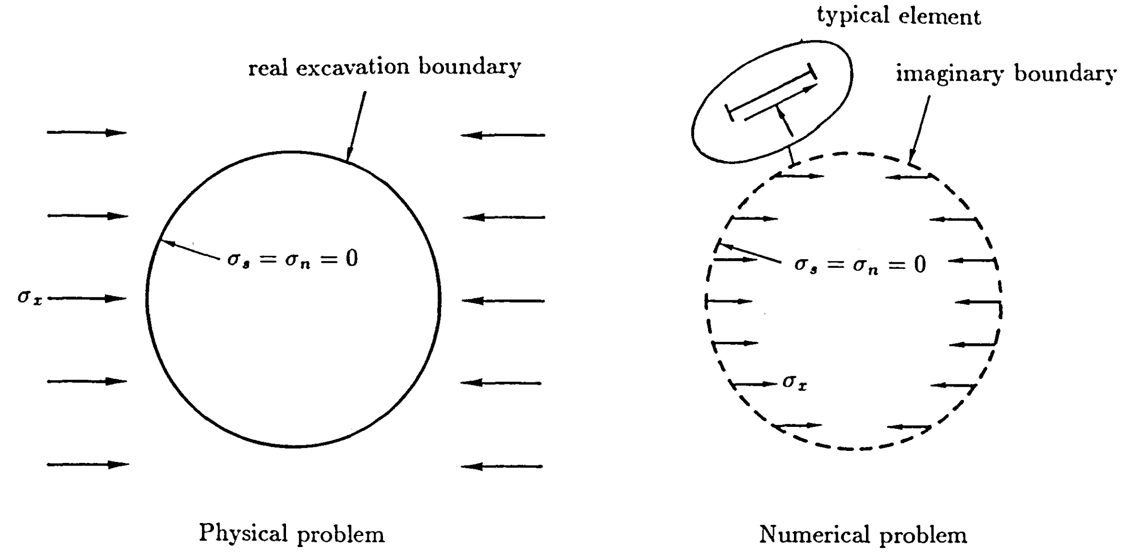


Figure 2.4 The boundary element method replaces the physical problem by an equivalent numerical problem.

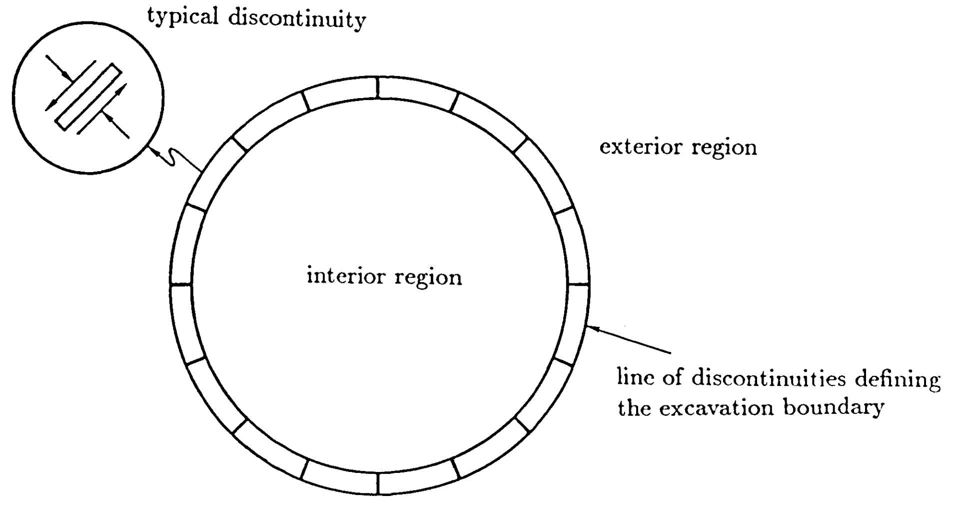


Figure 2.5 Displacement discontinuities used to define an excavation boundary.

The *displacement discontinuity* method (Crouch, 1976) is very similar to the boundary element method described above. Again the original boundary is replaced, this time by an imaginary cut which is divided into segments (Figure 2.5), each having unknown shear and closure displacements between the two faces. Influence functions, relating the magnitude of the displacement discontinuities to stresses along the imaginary cut, can be derived and used to develop equations that satisfy the original boundary conditions.

The displacement discontinuity method can also be used to represent real discontinuities in a rock mass. The boundary condition is then one of interaction between the two surfaces of a discontinuity and the displacements have real physical significance. This idea can be extended to the modelling of thin, vein like orebodies (Figure 2.6).

The *boundary integral equation* method is a direct method, and is similar to the boundary element method, except that use is made of Betti's reciprocal theorem to eliminate fictitious forces and displacements. Calculations can then be carried out in terms of real force and displacement distributions around the boundary.

Much of the work on the application of the boundary element method to problems in mine design has been conducted at the Royal School of Mines, Imperial College (Hocking, 1977; Brady, 1979; Hoek and Brown, 1980). In all these studies the rock mass was assumed to be an elastically homogeneous continuum. This restricts the use of such boundary element programs to mine design where the rock mass is hard and generally massive in nature. Recent developments in non-linear analysis using the boundary element method have been reported, *e.g.*, Banerjee *et al.* (1979) and Plischke (1988), but the intrinsic simplicity of the boundary element method is often negated. Another advance in the boundary element method has been the introduction of higher order boundary elements, such as Hermitian cubic elements (Watson, 1986), to model complex boundary geometries.

2.3.2 Plane Strain and Complete Plane Strain

Most two-dimensional boundary element programs that have been developed assume plane strain conditions, whilst the rest assume plane stress conditions. Plane strain means that the strains are restricted to a single plane. Hocking (1977) used the three-dimensional boundary element program described by Lachat and

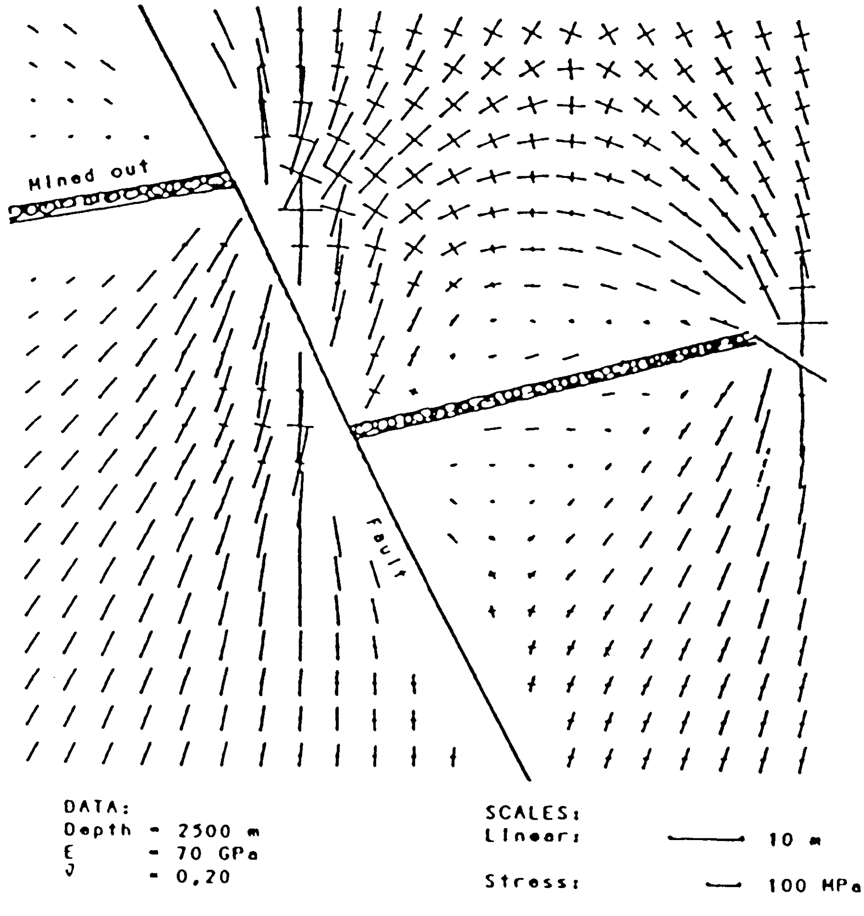


Figure 2.6 Displacement discontinuity analysis of a vein deposit intersected by a fault (after Brummer, 1988).

Watson (1976) to determine the stress distribution around openings with various length/cross-section dimension ratios. Hocking showed that when the ratio of the length to cross-section dimension exceeded 2.5, the stress distribution around the opening, at the centre of the section, approached that for plane strain. Plane strain analysis was also shown to overestimate stress magnitudes, giving an upper bound, which is important with regard to open stope design, as the design will tend to be conservative.

An implicit assumption in standard plane strain analysis is that one of the *in situ* principal stresses acts parallel to the long axis of the excavation. In general this is not the case, and excavations often occur inclined at an arbitrary angle to the triaxial stress field. In the complete plane strain method, the problem is decoupled so that there are two components – the plane problem (the problem considered in conventional plane strain analysis), and the antiplane or out-of-plane problem. Figure 2.7 shows the plane and antiplane stresses for an excavation.

Brady (1979) found that for a range of principal stress ratios which might be encountered in practice, the anti-plane stress component becomes significant when the axis of the excavation is inclined at an angle greater than 20° to an *in situ* principal stress. Complete plane strain analysis can be used in such situations, as long as plane strain conditions apply, otherwise three-dimensional analysis is necessary.

2.3.3 Comparison of Finite Element and Boundary Element Methods

Boundary element methods have several advantages over finite element methods for open stope design, as well as some disadvantages (Table 2.1). The main advantage of the boundary element method for open stope design is that the far-field boundary conditions are modelled correctly, and discretisation errors are restricted to the problem boundary. Stresses and displacements vary in a fully continuous manner throughout the problem domain. This is in contrast to the finite element method, where it is necessary in open stope design to define an arbitrary outer boundary to the problem domain. The problem domain has to be fully discretised in order to satisfy the far-field boundary condition of zero induced displacement. Since the problem domain has to be extended for quite some distance, in order to satisfy the far-field boundary conditions, the resulting problem size tends to be large, which leads to slower execution times, compared to boundary element

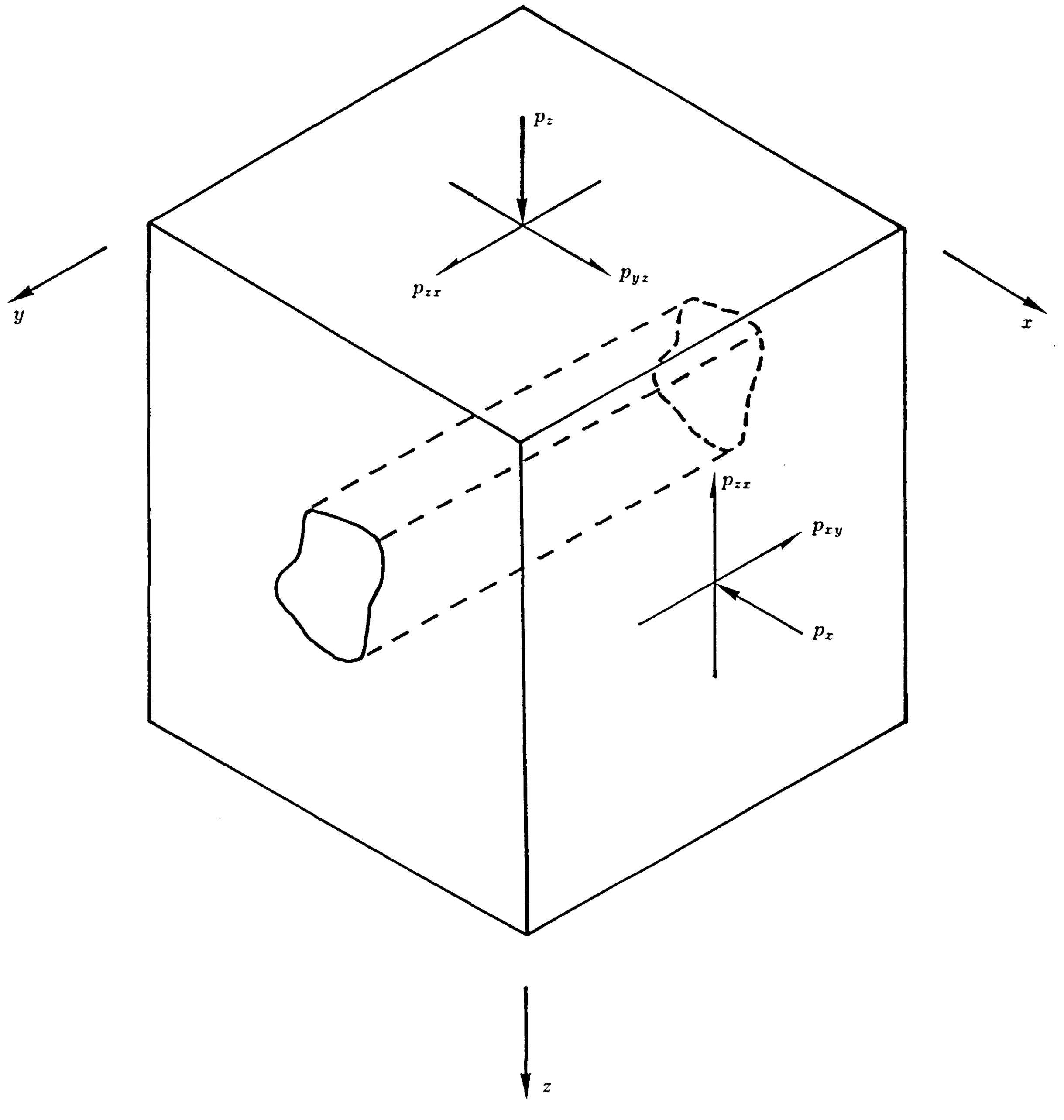


Figure 2.7 Plane (p_x, p_z, p_{zx}) and out-of-plane (p_{xy}, p_{yz}) stress components for a long opening excavated in a medium subject to a triaxial state of stress (after Brady, 1979).

Boundary Elements	Finite Elements
1. Discretisation errors restricted to the boundary.	1. Discretisation errors occur throughout the problem domain.
2. Field variables of stress and displacement are obtained directly.	2. Field variables of stress and displacement obtained by numerical differentiation
3. Field variables calculated only at points of interest.	3. Field variables calculated at all mesh points.
4. Non-linear and heterogeneous material properties difficult to model.	4. Non-linear and heterogeneous material properties simple to model.

Table 2.1 Comparison of the finite element and boundary element methods.

methods. This difficulty has been overcome to some extent by the development of infinite elements (Beer and Meek, 1981), which include in their formulations a decay term. Another solution to the modelling of an infinite domain has been the coupling of finite element and boundary element methods to produce hybrid programs (Section 2.5).

Boundary element methods, in general, are suited to linear material behaviour and homogeneous material properties. Although boundary element programs have been developed for non-linear and heterogeneous materials (Banerjee and Mustoe, 1978; Crouch, 1979) the intrinsic simplicity of the boundary element solution procedure is lost. The finite element method, however, can be more easily adapted for modelling problems involving non-linear material behaviour, including plasticity and heterogeneity. Joint elements, which allow for non-linear stiffness and shear strength characteristics of discontinuities to be modelled, have been developed recently (Brown, 1987).

2.3.4 Application of Boundary Element Method

Salamon (1964) described the face element method, which Plewman *et al.* (1969) used to produce the MINSIM program for analysis of stresses and displacements around deep tabular gold deposits in South Africa. In an extension to this, Starfield and Crouch (1973) developed a three-dimensional displacement discontinuity program for modelling inclined, planar, tabular seams. They demonstrated that the program could be used for the planning of excavation layouts, to determine which would result in the 'best' conditions of excavation stability. Batchelor and Kelly (1975) used the same program to model the tabular orebody at Wheal Jane mine, U.K. They also demonstrated an early form of back analysis whereby displacement measurements taken of the actual orebody were used to adjust the elastic moduli of the rock mass, so that the computed displacements matched the measured displacements. They found that *in situ* moduli had to be 10 to 50 times smaller than the laboratory values to enable realistic displacements, with sensible stresses, to be obtained. Once the model had been calibrated against existing workings, it could be used for planning stopes at deeper levels. Sinha (1979) developed a three-dimensional displacement discontinuity program at the University of Minnesota that Golder and Associates developed into NFOLD. NFOLD was a further extension in that more than one parallel, folded orebody could be analysed. NFOLD has been used extensively at Mount Isa mine, Australia for open stope design (Mathews *et al.*, 1983). Yu *et al.* (1983) describe the three-dimensional displacement discontinuity program MINTAB which has been used successfully in open stope design in Canada (Bawden and Milne, 1987).

Two-dimensional plane strain boundary element programs have been used widely in stope and pillar design. Brady (1977) used the program in Hoek and Brown (1980) to model the behaviour of two pillars in an open stoping block at Mount Isa mine, Australia. Brady (1977) showed that although an elastic continuum is intuitively an inadequate representation of a moderately jointed or stratified rock mass, the program could be used successfully provided that the rock mass failure criterion was obtained by back analysis. This failure criterion did not represent a fundamental mechanical property of the rock mass, but results were sufficiently consistent to allow the acceptance of the pillar design procedure. Lover *et al.* (1983) used a similar two-dimensional elastic plane strain boundary element program to undertake analysis of crown pillars. Brady (1979) developed an indirect boundary element program, TAB4, for modelling narrow excavations with complete plane strain. Brady demonstrated its use and applicability in open stope design, but it has not been used widely in practice. Crouch (1976) used the displacement discontinuity method to produce a two-dimensional plane strain program, MINAP. MINAP was able to model thin tabular orebodies, that had different rock properties to those of the country rock, as well as modelling faults and discontinuities.

Three-dimensional displacement discontinuity programs like MINSIM, MINTAB and NFOLD cannot give a three-dimensional stress distribution within the orebody, only at points away from the orebody or on the surface of the orebody. This is because the displacement discontinuity method assumes the orebody to be infinitesimally thin, and so the stress distribution is constant across the orebody. For this reason when stress distributions are required in pillars, conventional indirect or direct methods must be used. Where the pillar cannot be considered to be amenable to plane strain analysis, then true three-dimensional boundary element programs must be used. Watson and Cowling (1985) used a three-dimensional boundary element program to model the 1100 copper orebody at Mount Isa mine, Australia, in order to dimension large open stopes, which were essentially cubic in shape. Hocking (1978) modelled a series of pillars in the same orebody at Mount Isa, using the same boundary element program (Figure 2.8). Hocking (1978) showed that although it was a gross idealisation to assume linear elastic

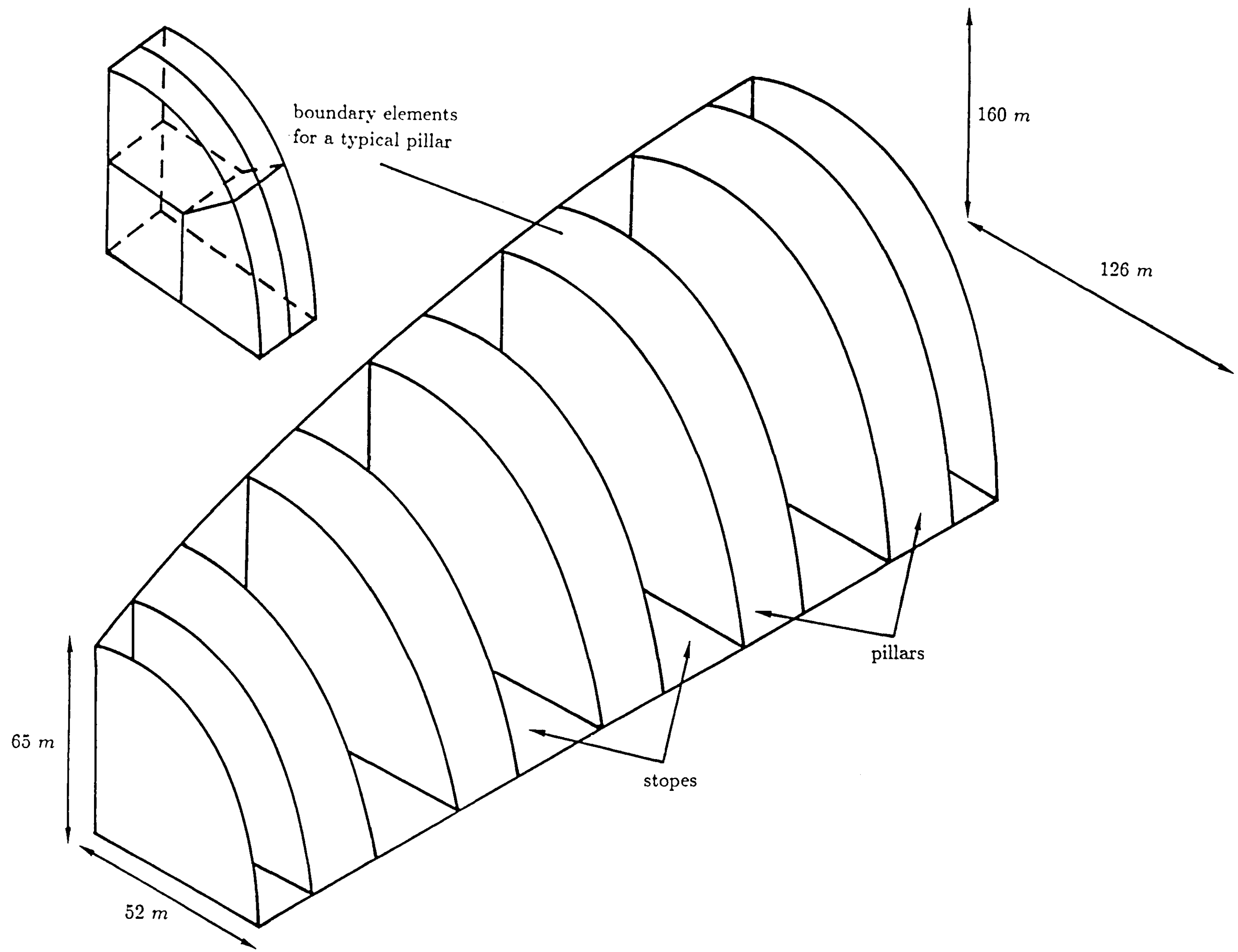


Figure 2.8 Boundary integral modelling of the 1100 orebody at Mount Isa mine, Australia (after Hocking, 1978).

behaviour for the pillars, qualitative results could be achieved in the selection of an extraction sequence from several alternatives.

2.3.5 Boundary Element Programs Implemented for the Research

A suite of boundary element programs were transferred to the college Cyber 960 mainframe computer and to an IBM XT micro-computer with mathematics coprocessor. They were assessed for their applicability for use in narrow, steeply dipping, tabular orebodies. Six two-dimensional programs and one three-dimensional program were tested and validated against classical elastic solutions (*e.g.* Sneddon, 1946; Savin, 1961; Jaeger and Cook, 1976).

The output from all the two-dimensional programs was standardised to that of BOUND, and the Hoek-Brown failure criterion added. A plotting routine for the two-dimensional programs on the micro-computer was developed in FORTRAN, that allowed contouring of σ_1, σ_3 and factors of safety, stress trajectory plots and failure mode plots. A similar facility was developed on the college mainframe computer using library plotting packages. MINTAB produces output that shows stress output normal to the plane of the orebody, but it is only graphical in nature. The Hoek-Brown failure criterion was not added to this program.

Below is given a description of the capabilities and restrictions of all the implemented boundary element programs.

BOUND

BOUND is a two-dimensional plane strain, fictitious stress, constant element, indirect boundary element program, given in Hoek and Brown (1980). BOUND was not designed for the analysis of orebodies with close parallel boundaries and requires considerably more elements than any of the other programs in order to properly discretise the boundary of an excavation. However BOUND, in its original form, can be used for horizontal as well as vertical sections through an orebody. This mode could be included in other two-dimensional programs, but vertical cross-sections in narrow tabular orebodies are more usual, as these orebodies tend to be worked at approximately 30 metre level intervals, but have considerable extent along strike.

BOUND is a conventional plane strain program which means that it assumes that the strain along the long axis of an excavation is zero. For this to be true the excavation has to be much greater in extent in this direction than in any of the other two directions. This generally precludes programs such as BOUND when analysing excavations that are basically cubic in shape. Such excavations have to be analysed using three-dimensional programs.

Conventional plane strain programs, like BOUND, also require that a principal stress direction is parallel to the long axis of the excavation. Brady (1979) found that, for the range of field principal stress ratios which might be encountered in practice, the anti-plane stress component becomes significant when the long axis of the excavation is inclined at an angle greater than 20° to a field principal stress.

TWOFS

TWOFS is a two-dimensional, plane strain, fictitious stress, constant element, indirect boundary element program, given in Crouch and Starfield (1983). It has the same advantages and disadvantages as stated for BOUND.

BEM11

BEM11 is a two-dimensional, complete plane strain, fictitious stress, constant element, indirect boundary element program, given in Brady (1979). BEM11 has the same advantages and disadvantages as BOUND and TWOFS for use in stope and pillar design, but it is a complete strain program. Thus, when the long axis of the orebody lies at an angle greater than 20° to a field principal stress, BEM11 rather than BOUND or TWOFS should be used.

TWODD

TWODD is a two-dimensional, plane strain, constant element, displacement discontinuity boundary element program, given in Crouch and Starfield (1983). The displacement discontinuity method is useful for the analysis of thin tabular orebodies because it represents the stope as an infinitely thin crack. There is need only to discretise one side of the stope – for example, the hangingwall. Thus the number of elements required to discretise each stope is much smaller than for the fictitious stress programs (BOUND, TWOFS and BEM11). However, the displacement discontinuity method, when used like this, is unable to give the stress distribution within a pillar, as it is constant across the pillar. This is due to the method itself, as in the formulation the pillar is assumed to have zero thickness.

In common with BOUND and TWOFS, TWODD is a conventional plane strain program and hence can only be used for stope design where the strike length of the stope is much greater than the down dip span, and where the long axis of the excavation is within 20° of a field principal stress. TWODD has the added advantage that it can be used to mimic TWOFS if the whole boundary of an excavation is discretised. Thus TWODD can act as both a fictitious stress and a displacement discontinuity program.

MINAP

MINAP is a two-dimensional, plane strain, constant element, displacement discontinuity boundary element program developed by Crouch (1976). Thus MINAP has the same advantages and disadvantages as TWODD, particularly with regard to the inability to model stress distributions within pillars, when used in normal mode. Like TWODD though, MINAP is able to act as a fictitious stress program and as a displacement discontinuity program and so can be used instead of TWOFS. MINAP has several additional features, not available in TWODD, but these slow the program down. Thus, if these features are not required, TWODD is a better program to use than MINAP. The additional features are:-

- Seam elements – these elements can have different geomechanical properties to the country rock (*i.e.*, Young's modulus, E, Shear modulus, G, and Poisson's ratio, ν). These seam elements can be mined or unmined and the mined elements can have a complete closure restriction which means that stress can be transferred across them if the hangingwall and footwall close together, such that they are touching.

- Linear and non-linear backfill elements – allowing the simulation of the filling of an excavation with backfill.

- Mohr-Coulomb elements - allowing the simulation of relative displacement and stress distribution around major discontinuities.

A major feature of MINAP is its ability to model a mining sequence (Figure 2.9). This is particularly useful with the Mohr-Coulomb elements as it allows the modelling of the effect of mining close to a fault – how the stresses and displacements change – as mining progresses.

TAB4

TAB4 is an indirect formulation of the boundary element method for narrow excavations and for complete plane strain, and was developed by Brady (1979). TAB4 is very similar in its capabilities to that of TWODD but is a complete plane strain program and so it takes into account the anti-plane component of stress. This means that it can truly model the situation where the long axis of an excavation lies at any angle to a field principal stress, which is highly desirable.

TAB4, in common with TWODD and MINAP, gives stresses within a pillar but they are constant across the pillar. This is due to the fact that the pillar is assumed to be of zero thickness in the method. For very wide pillars the stress distribution within a pillar does approximately become constant, but generally it cannot be considered as such. TAB4, as with the other two-dimensional plane strain programs, requires that the long axis of the excavation is much greater than any of the other dimensions.

MINTAB

MINTAB is a pseudo three-dimensional, displacement discontinuity boundary element program. As it is based on the displacement discontinuity method it assumes that the orebody is of zero thickness, and hence is unable to give the three-dimensional stress distribution within a pillar. However, it is able to give the stresses acting normal to the plane of an orebody. The program can only model one plane, so where orebodies occur that are parallel and closely spaced, other three-dimensional programs have to be used.

MINTAB is useful for modelling multiple stope mining in one vein, using a longitudinal section, as it can incorporate backfilling and there is the capability of sequencing, as with MINAP. MINTAB is good for assessing whether nearby stopes affect the stress distributions around one another, for showing how stress is thrown

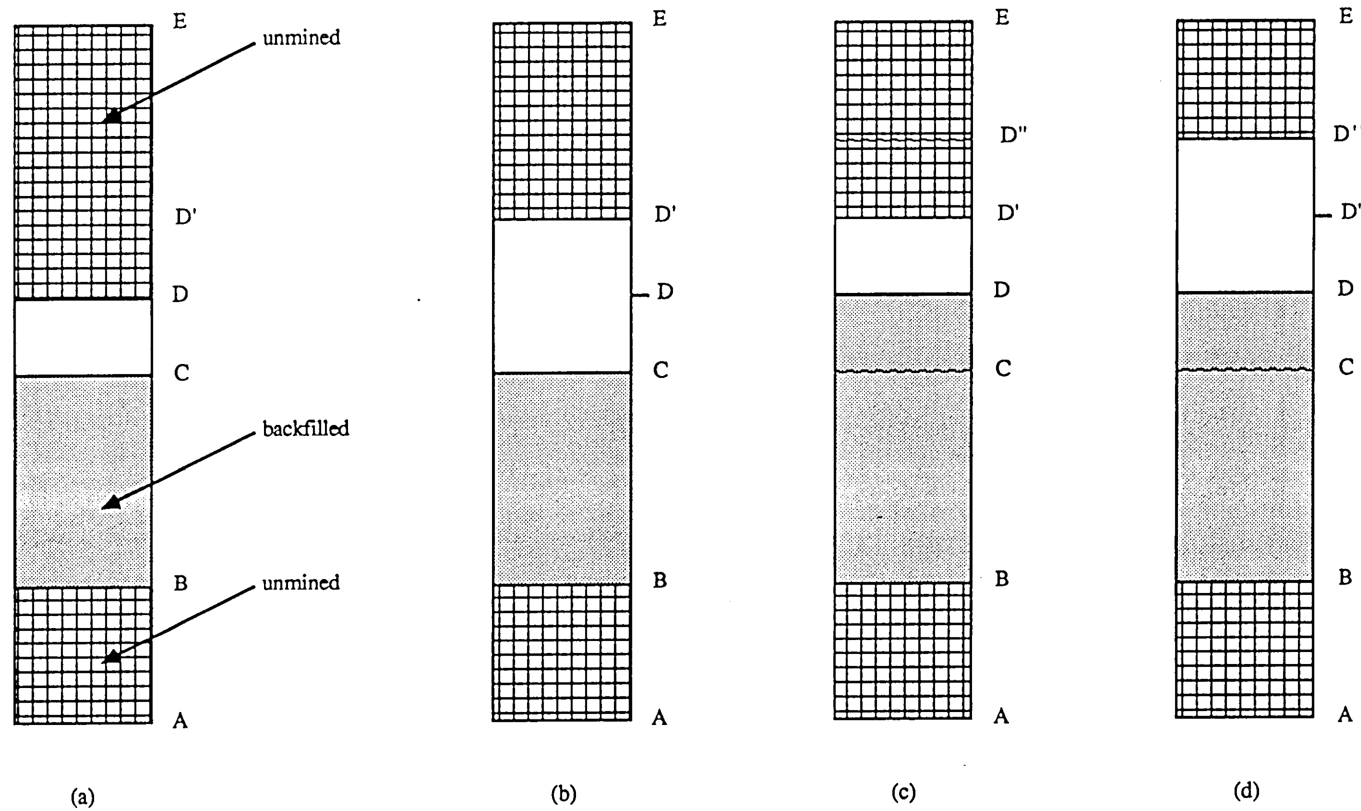


Figure 2.9 Modelling of a mining sequence: (a) initial state; (b) section DD' is mined; (c) section CD is backfilled; (d) section $D'D''$ is mined.

to the abutments of a mining block, and for determining whether a stope block is able to be modelled by a two-dimensional plane strain program, or whether it requires three-dimensional modelling.

Conclusions

Each of the programs has its own particular use in stope design. For the design of crown pillars, fictitious stress methods may be used in order to assess the stress distribution within the pillar. Where conventional plane strain analysis is acceptable, TWOFS is the preferred program (or TWODD used in its fictitious stress mode). Where complete plane strain analysis is required BEM11 is the preferred program. For determining zones of overstressing and tension around narrow stopes, TAB4 is the best program because it is a complete plane strain program. Where conventional plane strain analysis can be used the combined use of MINAP and MINTAB allow the mining sequence of a complicated system of stopes to be modelled.

2.4 Distinct Element Method

2.4.1 Introduction

Both the boundary element and finite element method can be modified to accommodate discontinuities such as faults and shear zones. However, any inelastic displacements are limited to elastic orders of magnitude by the analytical principles used to develop the solutions. Where the problem domain is such that it involves the interaction of finite numbers of discrete blocks of rock and the ratio of block size to the size of the problem domain is such that equivalent continuum behaviour cannot be assured, then special computational schemes are necessary.

The most powerful and versatile method available for simulating discontinuum behaviour is the *distinct element* method (Brady and Brown, 1985). The distinct element method was first proposed by Cundall (1971) as a means of modelling the progressive failure of rock slopes. The method used an explicit finite difference procedure and the dynamic relaxation technique. Stewart (1981) used a static relaxation procedure, similar to that introduced by Southwell (1940), as an extension to the dynamic relaxation based method developed by Cundall (1971). The distinct element method has been subsequently developed and applied by Cundall (1979) and Hart *et al.* (1988).

2.4.2 Methodology

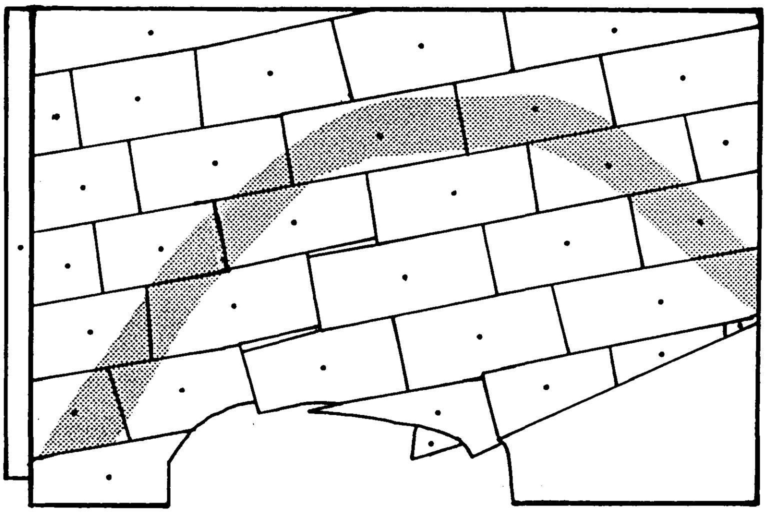
The distinct element method treats the rock mass as an assembly of blocks, interacting through deformable joints of definable stiffness. The stresses and displacements are continuous within each block, but are generally discontinuous between the blocks. In the original formulation of the distinct element method, the blocks are taken to be rigid and the deformations are associated with the contacts between the blocks. The computational scheme uses a dynamic relaxation technique to follow the motion of blocks through a series of increments of displacements, controlled by the time stepping iteration. Newton's laws of motion are used to determine the forces between, and the displacements of the blocks, during the progressive, large-scale deformation of the rock mass.

The distinct element method has been developed further from the original modelling of rigid blocks to the analysis of cracking of blocks and the analysis of the fully deformable block (Vargas, 1982). The program UDEC (Universal Distinct Element Code) has been developed to include all the previous capabilities for modelling various rock mass features (Lemos *et al.*, 1985). Work is still underway on the development of three-dimensional distinct element programs. Hart *et al.* (1988) described the first attempt, 3DEC. The updated version permits the modelling of elasto-plastic behaviour of intact blocks, various joint responses and other influences such as structure elements, fluid interactions, infinite domains *etc.*

2.4.3 Application of Distinct Element Method

Most of the applications of the distinct element method have been for the purpose of demonstrating the applicability of the numerical method. Although the distinct element method has not been used with notable success in design analysis, it has led to an improved understanding of aspects of discontinuous rock mass behaviour. Voegele (1978) used the method for tunnel design. Voegele modelled two tunnels in slightly different geological settings (Figure 2.10). In one case the tunnel was self-stabilising as two stress arches formed. One stress arch supported the material immediately above the tunnel and the other provided general support. The second tunnel had no local arch present, and tunnel support would be required to carry the weight of all the material below the main arch.

STABILISED ROOF



FAILING ROOF

Figure 2.10 Arching above an excavation in a blocky rock mass (after Voegele, 1978).

Stewart (1984) simulated open stope mining in a rock mass characterised by a primary continuous discontinuity set (*e.g.*, a bedding plane) and a secondary off-set joint set. Stewart modelled single stopes and stopes mined adjacent to each other, to examine the behaviour of the the rib pillars. Beer *et al.* (1983) used the distinct element method to model the bedded shale hangingwalls of open stopes in the lead-zinc orebodies at Mount Isa mine, Australia. The results given by the method did not agree with the observed behaviour of the hangingwalls. Beer *et al.* (1983) attributed this mainly to the artificial and unrealistic boundary conditions that they introduced close to the free surfaces.

Sutherland *et al.* (1984) used the distinct element method to model surface subsidence due to mining. Figure 2.11 (a) shows the distinct element model of the rock mass, with jointed rock blocks simulated by various shapes of elements. Modelling of excavations was carried out by removing the support blocks at the bottom and Figure 2.11 (b) shows the deformation after 7 blocks had been removed.

2.4.4 Conclusions

The distinct element method is the only method presently available for the analysis of a blocky rock mass subject to large displacements. Its lack of application to date is mainly due to practical reasons: large assemblies of blocks, such as are needed to model a block caving system, require large computer memory space and program run times tend to be very large. With the ever increasing speed of micro-computers and the rapid advances in super-computer technology, such practical constraints will disappear. The distinct element method has clear advantages over the other numerical methods for modelling blocky rock masses, and will increasingly be used in underground excavation design.

2.5 Hybrid Methods

2.5.1 Introduction

The advantages and disadvantages of the finite element, boundary element and distinct element methods have been discussed above. In many design analyses, a simple conceptual model of the rock mass can be devised allowing, in many cases, the boundary element method to be used, particularly if a parameter study is being conducted to assess various stope layouts. However, the simplification that

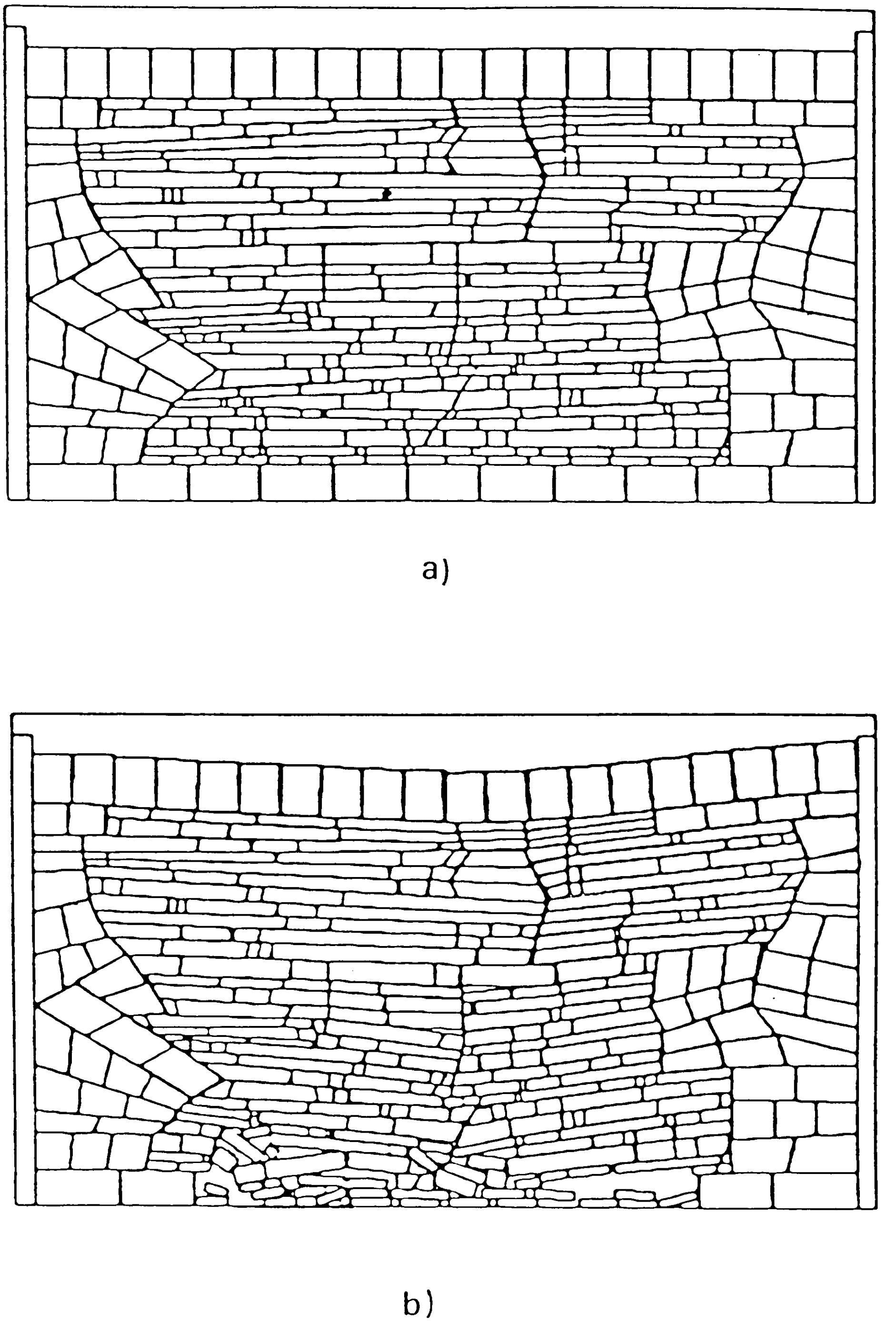


Figure 2.11 Modelling surface subsidence: (a) the distinct element mesh; (b) simulated strata movements (after Sutherland *et al.*, 1984).

occurs in such simple models can result in a loss of rigour in the design analysis. By linking several computational schemes the advantages of each method can be harnessed and the rock mass can be more realistically modelled.

The far-field rock, remote from an excavation, experiences only small deformations and can reasonably be represented as an elastic continuum. The near-field rock may experience much larger deformations associated with slip and separation at joint surfaces, as well as rigid block translation and rotation. Various codes have been developed which model the far-field rock using boundary element formulations, and the near-field rock with the appropriate differential method of analysis. A domain of complex behaviour is thus embedded in an infinite elastic continuum.

2.5.2 Finite Element - Boundary Element Method

Brady and Wassyng (1981) combined the boundary element method for modelling a homogeneous elastic region and the finite element method for an inhomogeneous region. Further development has been described by Yeung and Brady (1982) and Meek and Beer (1984). Beer (1985) described the coupling procedure, whereby the partial differential equations and the boundary conditions of the problem are reduced to a set of equations involving both finite element and boundary element discretisations. Meek and Beer (1984) described the use of a coupled finite element - boundary element program for the design of a closely bedded shale hangingwall of a stope at Mt. Isa mine, Australia (Figure 2.12). Beer *et al.* (1987) reported the development and application of a three-dimensional coupled program.

2.5.3 Distinct Element - Boundary Element Method

Lorig and Brady (1982, 1984) described a hybrid distinct element - boundary element program. The far-field rock was modelled using boundary elements, whilst the near-field rock was represented by distinct elements. This allowed different modes of rock response in the rock mass surrounding an excavation and in the support.

The linkage between the boundary element and the distinct element programs is different to that for coupled boundary element - finite element programs. The program proceeds by satisfying the displacement continuity and equilibrium conditions at the interface between the two domains (Brown, 1987). In other words it was assumed that no slip or separation could occur at the interface.

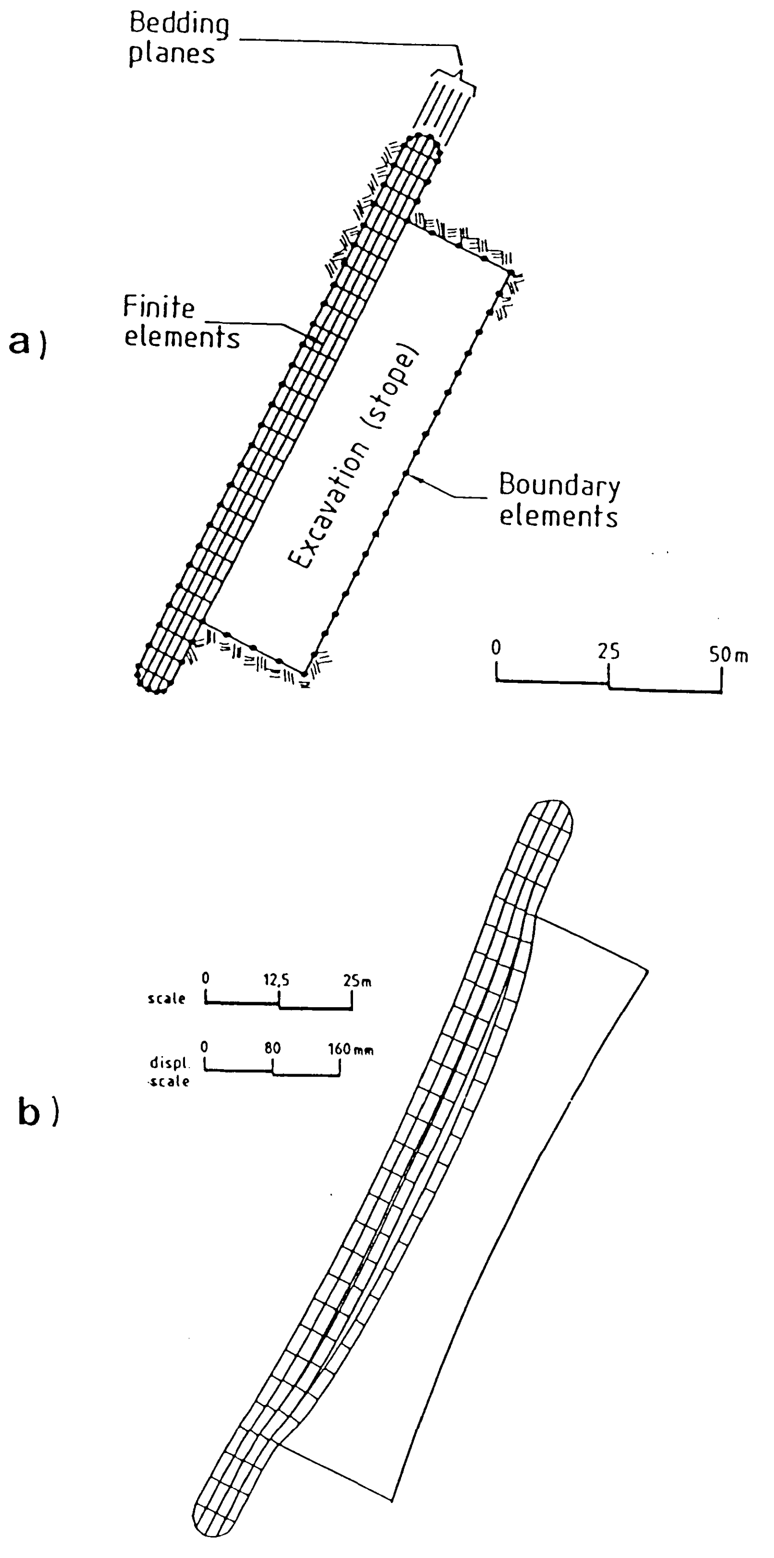


Figure 2.12 Example of a hybrid finite element – boundary element analysis of an underground excavation: (a) finite element and boundary element meshes; (b) displaced shape showing slip and separation on bedding planes (after Meek and Beer, 1984).

As Brady and St. John (1982) state, validation of such programs is difficult. They concluded that field experiments can make virtually no contribution in program validation for programs which incorporate non-linear constitutive behaviour. To date no practical use of such codes has been reported in the literature, but Lorig and Brady (1984) and Brown (1987) give schematic examples of the use of such programs. Figure 2.13 shows a schematic diagram of the use of a distinct element – boundary element program in underground excavation design.

2.5.4 Finite Element - Distinct Element Method

Little work has been reported in the literature, apart from Dowding *et al.* (1983) and Pan (1989). Dowding *et al.* (1983) presented a coupled finite element - rigid block model to analyse lined openings in a jointed rock mass. Plischke (1988) presented a survey of computer programs in rock mechanics research and engineering practice and no coupled finite element - distinct element programs were given. Pan (1989) developed a program COUPLE and demonstrated its use in the analysis of gate roadways at Coventry Colliery, U.K. Pan (1989) showed that the hybrid program is capable of dealing with the most general analysis of strata movements and in assisting with the design of mining excavations.

2.5.5 Conclusions

The purpose of hybrid programs is to model the far-field rock mass with boundary elements (as a continuum) and the the more complex constitutive behaviour of the near-field rock with finite elements (as a pseudo-continuum) or distinct elements (as a discontinuum). The advantages of this approach are that uncertainties associated with the assumption of an outer boundary for the problem, as required by the differential methods, are removed and also the far-field and elastic material behaviour is represented in a computationally and mechanically appropriate way with boundary elements.

Such programs attempt to model the mechanisms that can be observed to be present around underground excavations. As such, they appear to have great potential for future numerical analysis in mine design. However, many of these models are still at the research stage and have yet to be validated or demonstrated on practical examples. Also, no methodology for the practical use of such programs has yet been established, and so it will probably be many years before they become design tools.

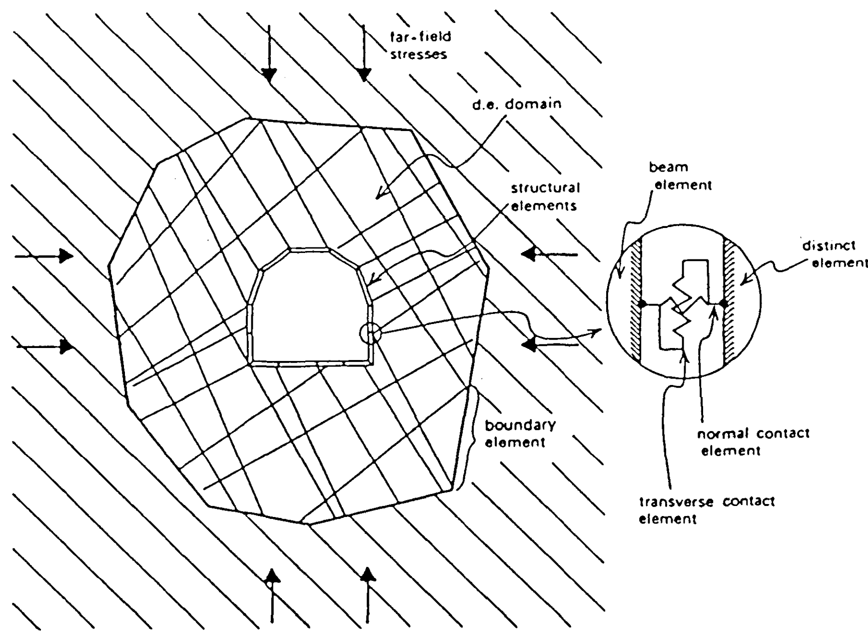


Figure 2.13 Example of a hybrid distinct element – boundary element analysis for underground excavation design (after Brady, 1987).

2.6 Conclusions

The discussion above has reviewed the numerical analysis techniques available for underground excavation design. It has been shown that all the techniques have advantages and disadvantages for use in the design of stopes and pillars. However, the boundary element method has clear advantages over the other methods, in its ability to model infinite problems, in its small data set required, and in its computational simplicity.

This research is concerned with open stope mine design in rock that is hard and relatively massive. More specifically, interest has concentrated on narrow, steeply dipping, tabular orebodies, which are a common and important mineral source. Such orebodies, when found at reasonable depth, can often be considered to be amenable to elastic continuum analysis. Also, Brady (1977) showed that moderately jointed or stratified rock masses could be modelled using boundary element programs, provided that the failure criterion was obtained by back analysis.

In Chapter 3, it will be shown that conceptual modelling, based on the mechanical behaviour of a rock mass, allows the correct type of analysis to be assigned to a design problem, and that in many cases elastic continuum analysis using boundary element programs is appropriate. An observational approach to stope design is proposed, whereby boundary element programs form part of an advanced design methodology for open stope and pillar design. Using this design methodology such design work should become a routine exercise for geotechnical engineers at mining operations.

With the rapid increase in computer technology, micro-computers have become as powerful as mainframe computers in the 1960's. This has enabled more complex numerical analysis programs to be run on microcomputers at mine sites. However, not all of these advanced numerical techniques have been validated for use in open stope design and as Brady and St. John (1982) noted, methodologies for their use have often still to be developed. These programs also tend to be quite complex to use, and in trying to model more of the complexity of the rock mass can often lead to results that do not reflect the failure mechanisms present.

Undoubtedly though, these advanced programs will play an increasingly larger part in mine design in the future. This research is not, however, tied to the principle that boundary element methods are always the correct numerical method to use, and in the conceptual models that are discussed in Chapter 3, and developed in Chapter 6, it is clearly defined when their use is deemed appropriate. In all other cases other numerical methods are recommended. However, common to the use of all these numerical techniques in rock mechanics design, is the adoption of an observational approach, in that our understanding of the behaviour of the rock mass develops as excavation proceeds. The design should be flexible enough to allow modifications to be made based on the results of instrumentation and observation.

CHAPTER 3
OPEN STOPE DESIGN

3.1 Introduction

In Chapter 1 the role of rock mechanics in mine design was discussed, and it was demonstrated that the engineering design of stopes was of vital importance. In this chapter the design of open stopes is critically assessed in order to determine the most appropriate stope design methodology, concentrating specifically on narrow, steeply dipping, tabular orebodies. Section 3.2 describes the open stoping mining method, and shows that with the large open stopes used in modern mines, good stope design is crucial in determining the success of an open stope layout.

Historically, stope design was by precedent practice but the use of numerical analysis in stope design is becoming more common. To date though, few clear methodologies for the use of numerical analysis programs in mine design have been developed. Rock mechanics modelling methodologies must take into account the uncertain and complex nature of a rock mass. For this reason the methodologies developed in other engineering disciplines for numerical analysis are inapplicable. Section 3.3 discusses rock mass modelling philosophies and proposes the use of conceptual models as a means of unambiguously describing a rock mass. The importance of understanding the mechanics of failure around open stopes is also noted.

Section 3.4 critically assesses the approaches to open stope mine design that have been identified in the literature and the approach of simple conceptual modelling is selected as the most appropriate for open stope design. This modelling approach utilises simple numerical programs, which were described in Chapter 2, and combines these with an observational approach. The design process is seen to be an iterative one, whereby the actual performance of the rock mass is monitored and compared to the computed behaviour. If necessary, the analysis is modified so that predicted behaviour matches observed behaviour.

Finally, in Section 3.5 an advanced methodology for the design of open stopes in narrow, steeply dipping, tabular orebodies is proposed. The methodology uses site characterisation data in order to develop conceptual models of the rock mass. For each conceptual model there is an appropriate numerical analysis technique. Where the behaviour of the rock mass can be approximated to that of a continuous, homogeneous, linear-elastic medium, then the simple elastic boundary element programs, that were identified in Chapter 2 as being of particular use in narrow, tabular orebodies, can be used.

3.2 Open Stoping Mining Method

Open stoping is one of the naturally supported mining methods which are characterised by small displacements in the country rock and the large strain energy stored in the near-field rock (Figure 3.1). Open stoping is the generic name given to the mining method, although terms such as open, sublevel, longhole and blasthole are used to describe particular methods within the group (Brown, 1985). Open stoping is applied in massive or steeply dipping stratiform orebodies. For an inclined orebody, which has inclined stope walls, the inclination of the stope footwall must exceed the angle of repose of the broken rock by a suitable margin, in order that the fragmented rock flows freely to the extraction level. Open stoping requires orebodies to have fairly regular boundaries, since selective mining is not possible due to the use of long blastholes. Figure 3.2 shows a schematic layout of an idealised underground metalliferous mine, and illustrates much of the mining terminology used in this and subsequent chapters.

Open stoping requires a stable hangingwall and footwall, as well as a competent ore, because the mining method produces unsupported, free standing stope walls and roof. There is little opportunity to apply systematic reinforcement to the stope boundaries, as is possible with cut-and-fill mining, so the ability of open stopes to remain locally stable for a short period of time is essential, if dilution of the ore reserves is to be avoided. Increasingly, cable bolting of stope walls and pillars is being practised, but as Bridges (1983) and Fuller (1983) noted, the method has only been marginally successful. Fuller (1983) found that overbreak occurred in 75% of the cable reinforced open stopes studied, ranging from 8% to 23% by volume. Fuller (1983) concluded that the density of cable bolts needed to be increased in order to improve hangingwall control and minimise dilution.

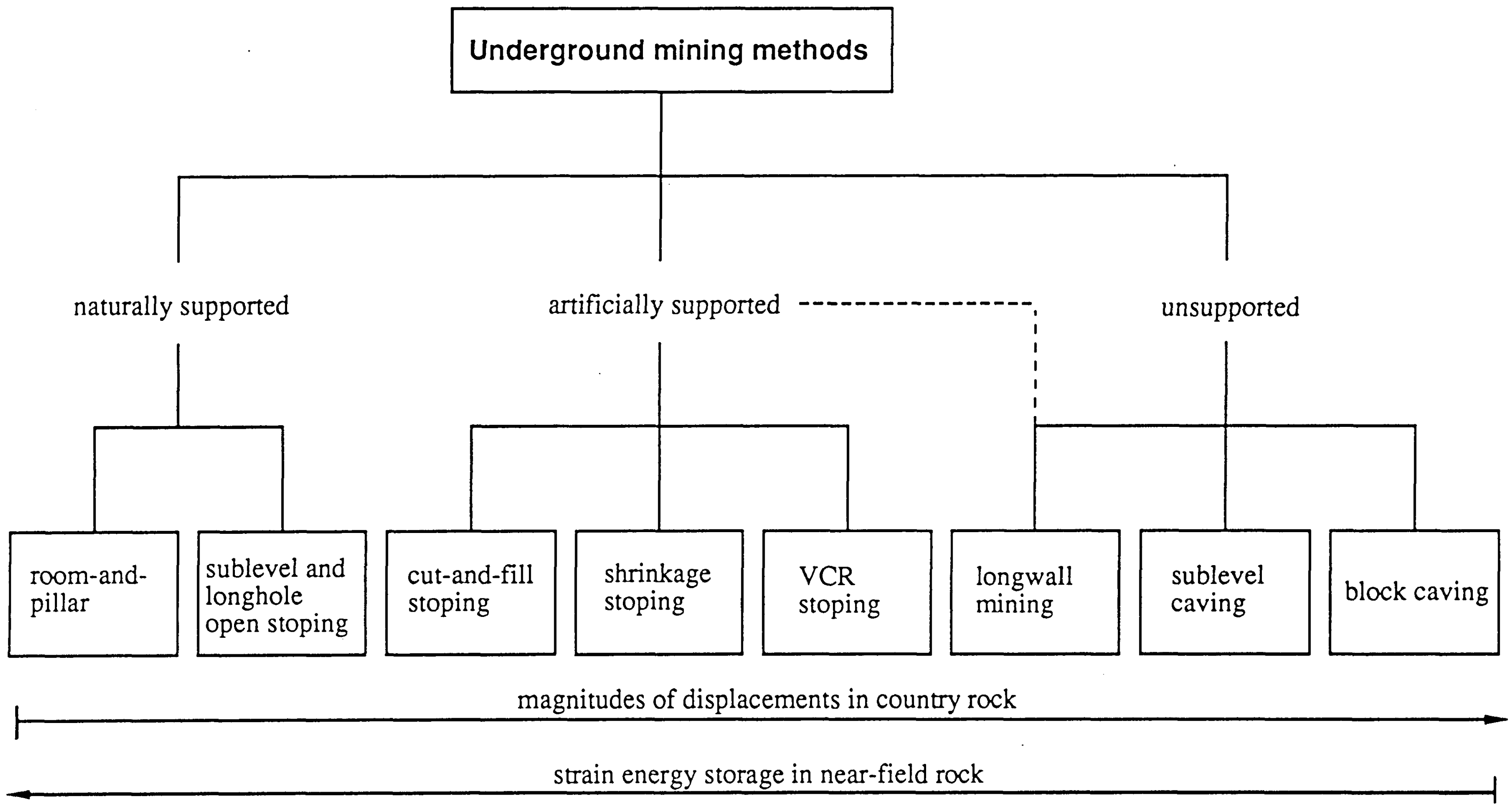


Figure 3.1 A hierarchy of underground mining methods and associated rock mass response to mining (after Brady and Brown, 1985).

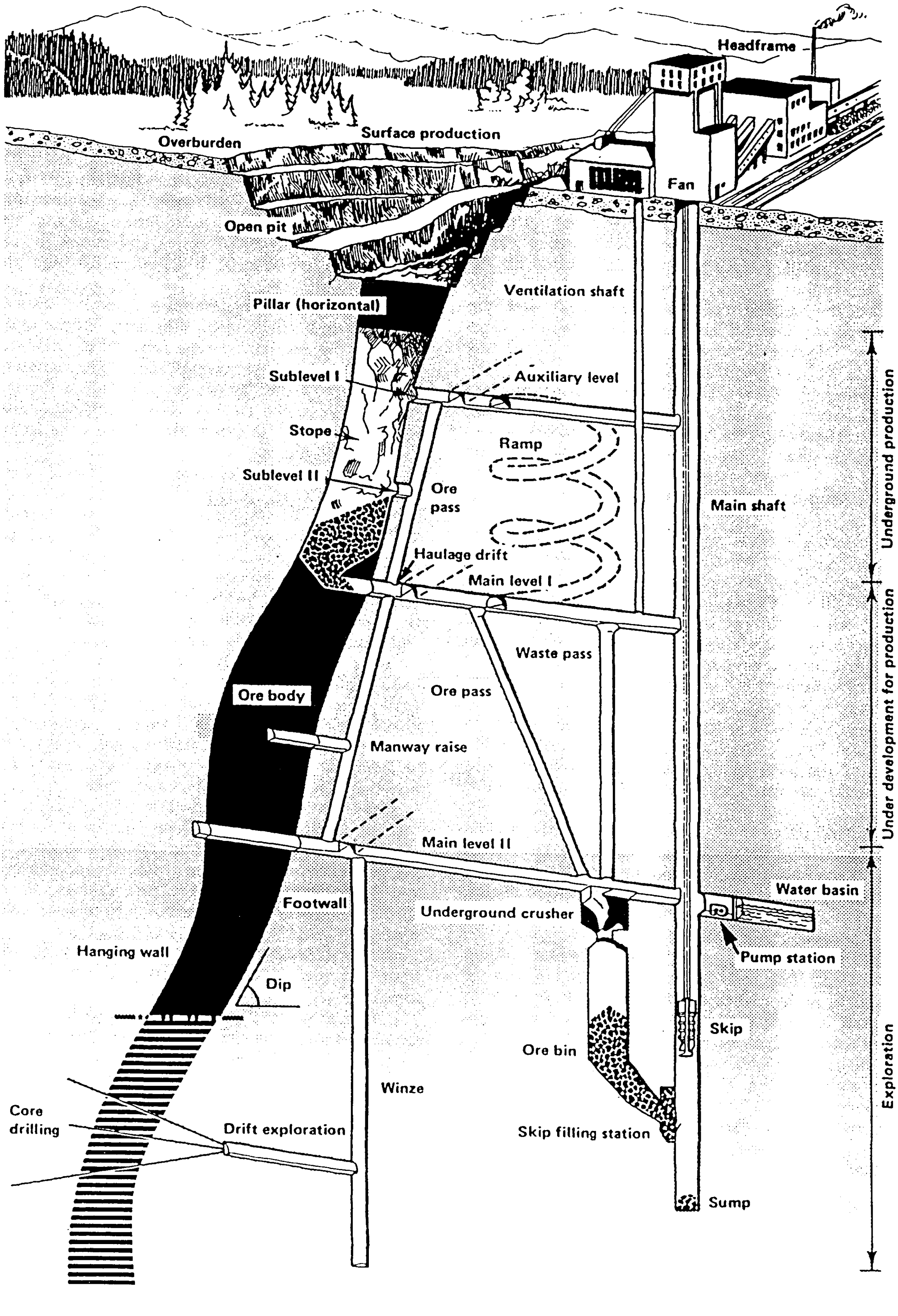


Figure 3.2 Schematic layout of a typical underground metalliferous mine (after Hamrin, 1982).

As Figure 3.3 shows the essential problem lies in achieving the required distribution of cables over the whole length of the hangingwall, especially where the open stopes are tall, from a normal drill drive. Figure 3.3 shows that the use of special hangingwall drill development allows more of the hangingwall to be covered, but such development is expensive.

Stope pre-production development is quite extensive, and consists of an extraction level, access raises and drifts, drill drifts, slot raise and stope return airway. Each stope layout is unique but Figure 3.4 shows a schematic layout for sublevel open stoping with ring drilling. A stope is begun by enlarging a slot raise by slashing (parallel hole blasting) to the width of the stope. Ore is then broken in the stope by blasting ring drilled or parallel hole drilled longholes, using the free face provided by the slot. Fan drilling is most common, although parallel drilling is preferred for narrow orebodies generally. The length of the blastholes does not normally exceed 25 m, with hole diameters being in the range 50–200 mm. The length of the blastholes is controlled by the drilling accuracy. The advent of down-the-hole (D.T.H.) drills, with the increased accuracy that they provide, has led to blastholes over 50 m in length, with parallel drilled holes being favoured.

Broken ore collects at the drawpoints and is recovered in a variety of ways. Traditional methods of rocker-shovel and tracked haulage are increasingly being replaced by trackless load-haul-dump (L.H.D.) machines. These L.H.D.'s collect the broken ore from the drawpoints and carry the ore along the access drives to be deposited in the ore-pass system. The drawpoint and access drive are usually situated in the footwall of the orebody.

In massive orebodies, the mining method is similar, although the stope width is not dictated by the ore boundaries, and stopes tend to be square in plan. Figure 3.5 is an example of the open stoping method used at Mt. Isa mine, Australia, where the massive copper orebody is mined in a checkerboard fashion. There the stopes are backfilled once all the ore has been removed, not in an attempt to support the stresses that the original rock supported, but to minimise displacements in the rock around the stope boundaries. Backfilling using sand, rock or cemented hydraulic fill is becoming increasingly common, especially at depth, as it enables a higher extraction ratio compared with that achieved when backfilling is not employed.

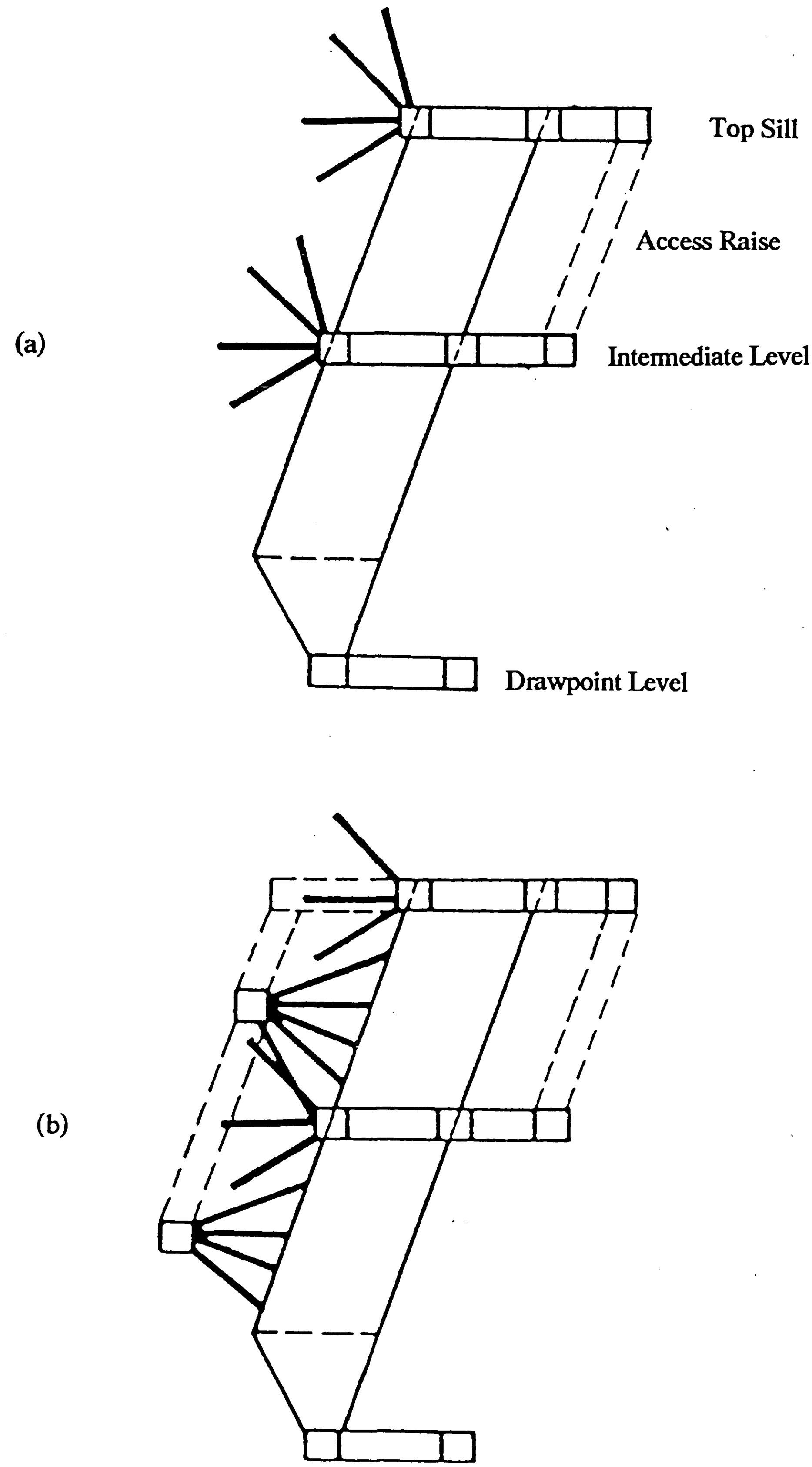


Figure 3.3 Cablebolting of open stope hangingwalls: (a) localised support with cables installed from hangingwall drill drives; (b) cable support from specially driven access and hangingwall drill drives (after Fuller, 1983).

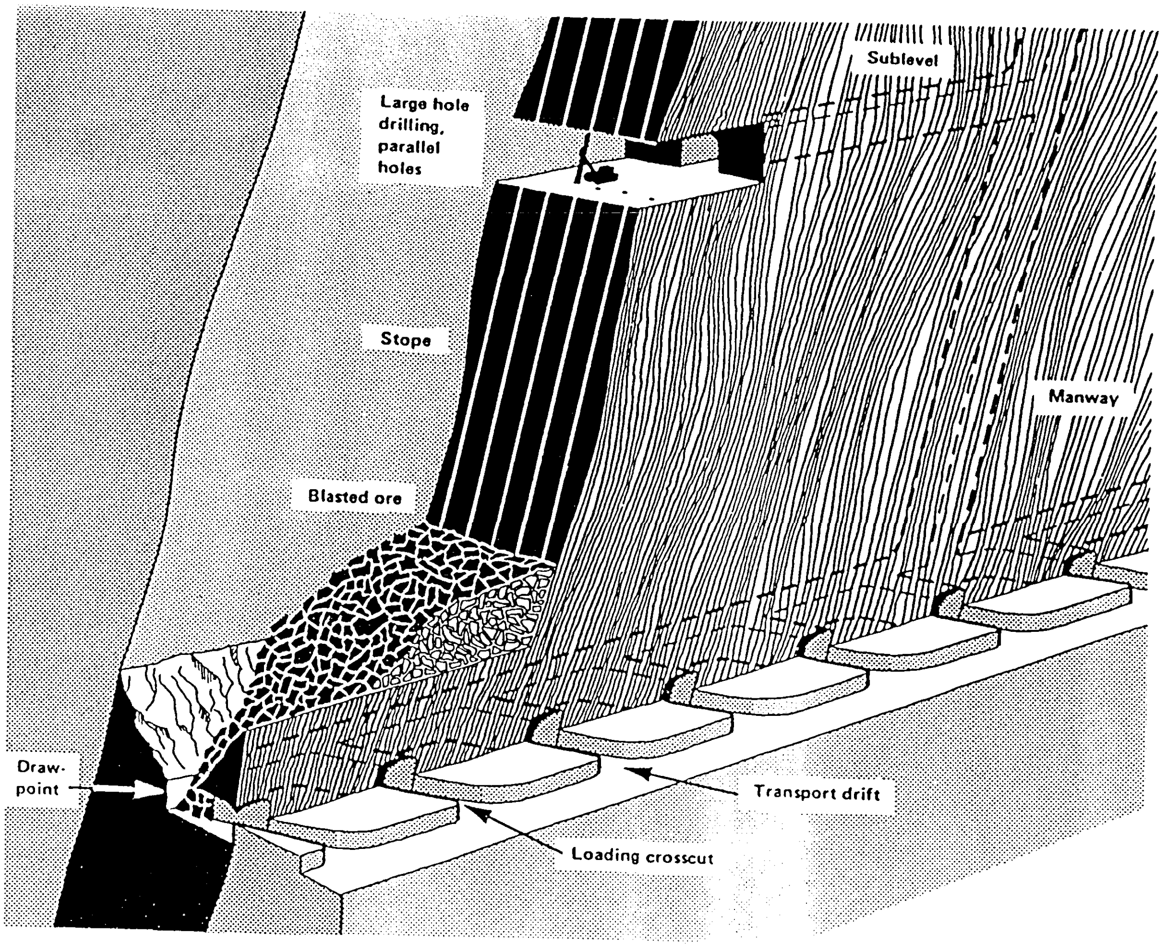


Figure 3.4 Sublevel open stoping with large-hole drilling and blasting (after Hamrin, 1982).

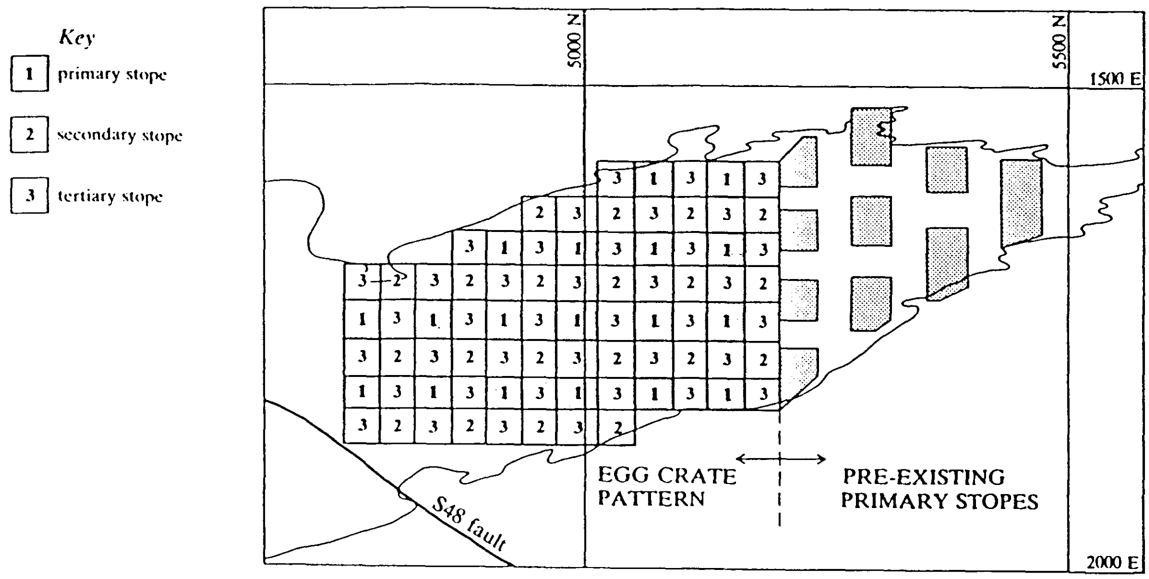


Figure 3.5 Plan view of ore extraction in the 1100 copper orebody, Mount Isa mine, Australia (after Alexander and Fabjanczyk, 1981).

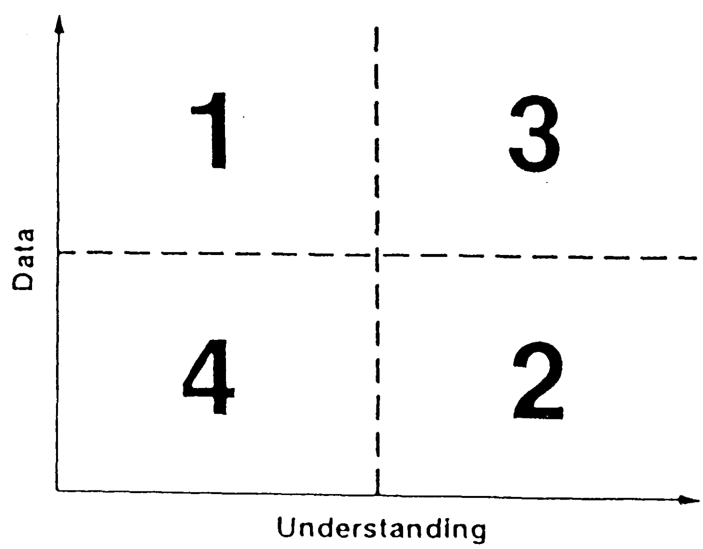


Figure 3.6 Holling's classification of modelling problems (after Starfield and Cundall, 1988).

Open stoping is now practised widely throughout the world. Bridges (1983) found that it accounted for 67% of the production from underground metalliferous mines in Australia in the early 1980's. Increased use of the method in the future is expected in Australia, Canada, Scandinavia, U.S.A. and Africa (Brown, 1985). In many cases it is replacing shrinkage stoping, or where fill is used, cut-and-fill mining, as the preferred mining method. This is because open stoping is a highly efficient mining method, adapted to use the latest mining technology in drilling and mucking. Open stoping also has the advantage that the parts of the mining cycle – drilling, blasting, mucking – can be carried out separately from one another. From a safety point of view, open stoping is relatively safe compared to shrinkage stoping or cut-and-fill mining, as it is a non-entry method, *i.e.*, personnel are not in the stope. Personnel are restricted to the drill drives and the drawpoints.

Open stoping does have the disadvantage that it requires a large capital outlay for the development work. However, as Lawrence (1982) noted productivity rates are high, equipment utilisation is high, dilution is low and ore recovery is usually around 100%. On a cost/tonne basis open stoping is one of the cheapest mining methods.

3.3 Rock Mass Modelling

3.3.1 Introduction

Rock mechanics emerged as an engineering science around thirty years ago, at about the same time as computers were becoming more available. The initial work in rock mechanics concentrated on rock properties and field measurements, as the available computational models, developed in other fields of solid mechanics, were perceived as being too simplistic, due to their lack of geologic detail. As numerical methods, notably finite element methods, were developed further in other engineering disciplines, programmers in rock mechanics started to develop more and more complex models. These models began to allow for a variety of material behaviours, and inclusions of materials with different properties to those of the surrounding rock. Programmers were spurred on to develop even more complex programs, incorporating more detail, in an attempt to incorporate all aspects of the rock mass.

However, a rock mass is not directly analogous to other engineering materials. In engineering the behaviour and properties of the materials are generally well understood, and the materials are generally continuous and homogeneous. A rock mass is not as simple as an engineering structure, and the *in situ* rock properties are difficult and expensive to measure. Thus what had been produced were models which were as complex as the rock mass itself, which were sensitive to variations in rock mass properties and boundary loading conditions, and for which no modelling philosophy had been developed. As Starfield and Cundall (1988) stated:

'*The art of modelling lies in determining what aspects of the geology are essential for the model. The challenge is to turn that art into a methodology.*'

3.3.2 Modelling Philosophy

Holling (1978) devised a classification that modellers in ecology have found useful (Figure 3.6). There are two axes, one measuring quality and/or quantity of data, while the other measures the understanding of the problem to be solved. This classification has four regions. In region 1 there is good data but little understanding and is the region where statistics is the appropriate modelling tool. In region 3 there is good data and also good understanding, and is where models can be built, validated, and used with conviction. Regions 2 and 4 relate to problems that are data limited, in that data is unavailable or cannot easily be obtained.

Many problems in engineering fall in region 3, whereas problems in rock mechanics usually fall into the data-limited category. It is seldom the case that enough is known about the rock mass to model it unambiguously. For nuclear waste disposal repository design, the approach adopted has been to try to collect sufficient data, so as to move the problem into region 3 (Whyatt and Julien, 1988). However, for general rock mechanics design, it is futile to expect to be able to gather sufficient data (Starfield and Cundall, 1988). The approach to take is to develop modelling philosophies for rock mechanics problems that lie in regions 2 and 4, using numerical programs that were developed for use in region 3. As Starfield and Cundall (1988) stated:

'*We must accept that rock mechanics does not fit the more conventional mould and develop a modelling philosophy that fits rock mechanics, instead of trying to fit rock mechanics to the prevailing philosophy.*'

3.3.3 Conceptual Models

Starfield and Detournay (1981) proposed the use of conceptual models as a means of providing a framework for the unambiguous description of a rock mass. They developed a preliminary set of conceptual models for tunnel support design. Starfield and Detournay (1981) realised that there is rarely a single theory that can be applied to all rock conditions but that there are a series of models, all with a restricted range of operation, which are of use in a number of different geological conditions. The purpose of building conceptual models is that it forces the modeller to associate *in situ* conditions with each of the geological models, with an emphasis on the mechanics of failure.

Starfield and Detournay (1981) also proposed a *two-pass* approach to site investigation. In the first pass, field data are used to select the appropriate conceptual model. Once the appropriate conceptual model has been identified, then in the second pass the key field parameters are related to the material properties of the model. The analysis and design work is then related purely to the selected model. Design alternatives appropriate to the model can be clearly identified, whilst the mechanical structure of the conceptual model also identifies the key field parameters. In Chapter 6 simple conceptual models of rock, based on easily measurable rock mass properties, have been developed for open stope design, and linked to these conceptual models are appropriate design procedures, depending on the actual design problem. These conceptual models and developed methodologies are specifically designed for open stopes in narrow, steeply dipping, tabular orebodies but the approach could be extended to all types of open stope.

3.3.4 Open Stope Failure Mechanisms

In order to build a conceptual model of a rock mass for open stope design, the failure mechanisms around open stopes have to be understood. Failure around open stopes is caused by the mining of rock to create an excavation, whereby the mining of rock removes its supporting effect on the rock mass adjacent to the excavation. Removal of this support also leads to stress redistribution around the excavation and to zones of compression and tension. The nature of the rock mass then dictates how these changes in *in situ* stresses and removal of support effect the performance of the rock mass.

A rock mass consists of blocks of rock of various sizes and shape between which are a variety of materials. The shape of the blocks present determines whether the rock mass can be considered to be isotropic or anisotropic, and the size of the blocks, relative to the surface of the rock mass exposed, is very important in determining the stability of the rock mass. Size of rock blocks is probably the most important factor influencing excavation stability because discontinuities are the weakest component of the rock mass. The smaller the blocks are, the more discontinuities there are exposed and hence the less stable will be the rock mass surface. Figure 3.7 shows how the size of the exposed surface effects the behaviour and characteristics of the rock mass.

There are basically three types of rock behaviour around open stopes, and with each rock behaviour type there are associated failure mechanisms.

Intact Rock Behaviour

The rock mass surrounding an excavation will behave as an intact rock when the relative size of the blocks, of which the rock mass is composed, are similar in size or larger than the excavation surface dimensions. For design purposes such a rock mass can be considered to be homogeneous, isotropic and linearly elastic. Two types of failure can occur, one due to compressive stresses, the other due to tensile stresses.

Failure can occur parallel to an excavation surface when compressive stresses exceed the rock mass strength. This leads to tensile cracking and slabbing of the rock (Figure 3.8*a*). Another commonly observed effect of high compressive stresses occurs at sharp corners of excavations, where slabbing of the rock can lead to a smoother profile being produced. In areas where tensile stresses are present, tensile cracks may develop perpendicular to the acting tensile stresses and create a zone of relaxation (Figure 3.8*b*).

Discrete Rock Behaviour

A discrete rock mass is defined as one that is not extensively fractured and contains three or less joint sets. The spacing of the joint sets are relatively large, which produces blocks of rock in the order of several cubic metres.

In an isotropic rock mass failure can occur through the shearing of wedges due to high compressive stress (Figure 3.9*a*). Failure can also occur in areas that

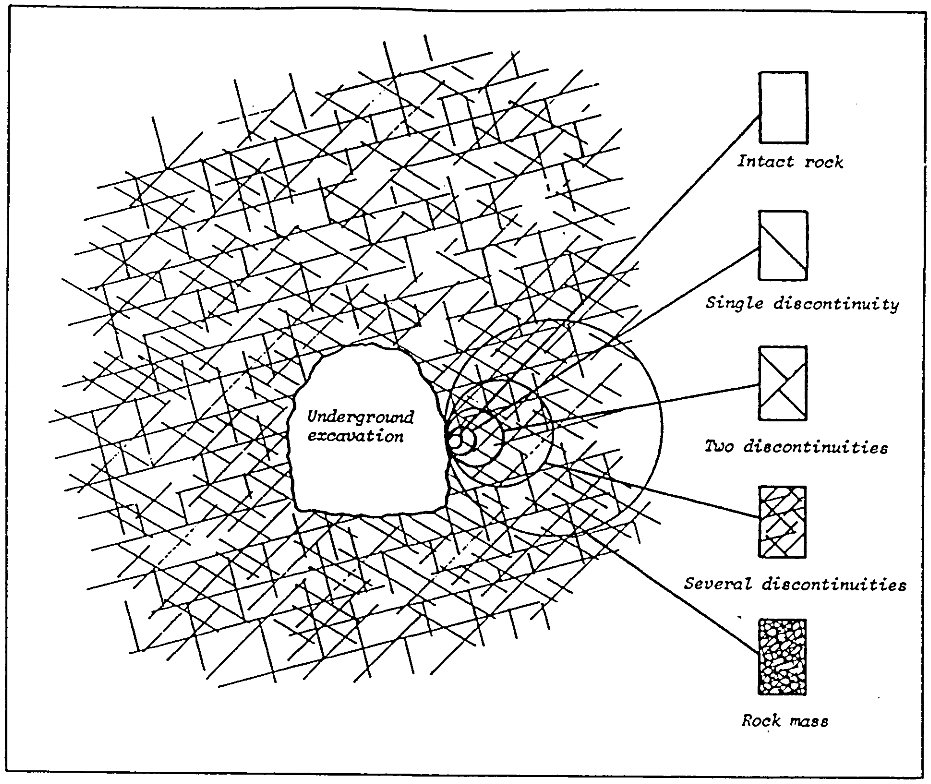
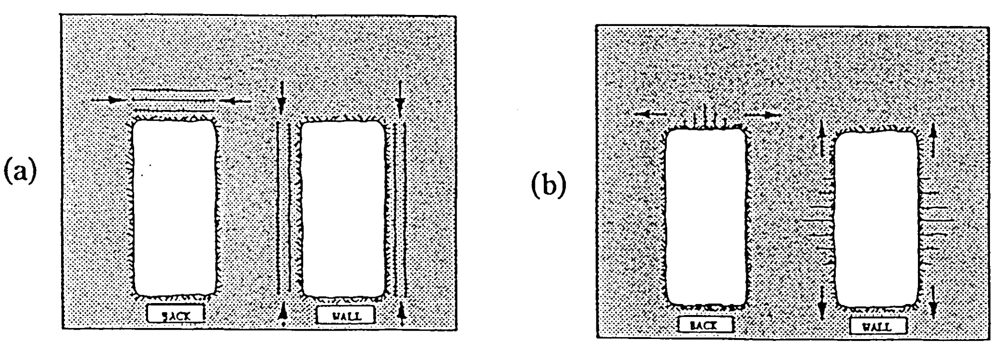


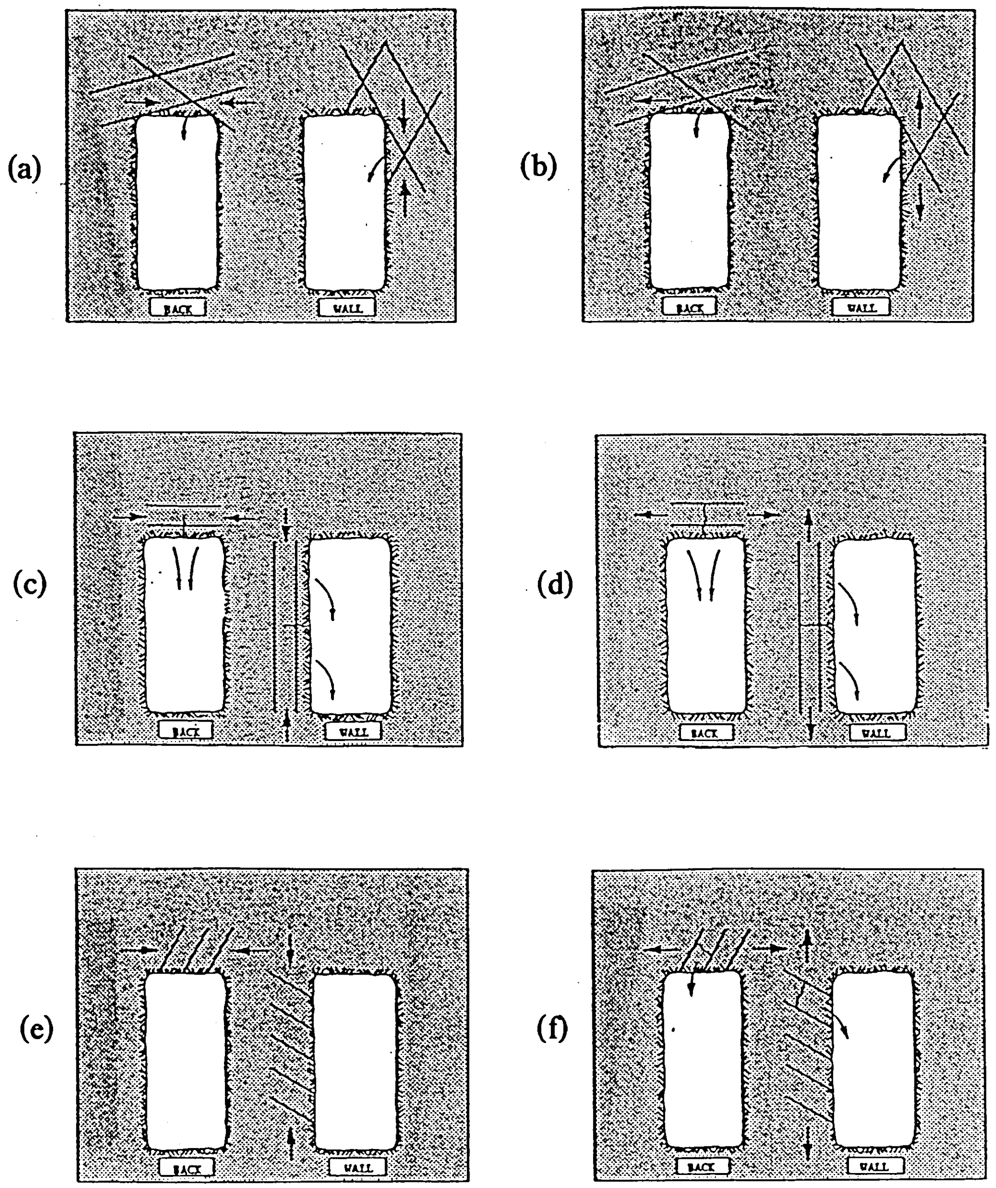
Figure 3.7 Idealised diagram showing the effect of sample scale on rock behaviour (after Hoek and Brown, 1980).



(a) Failure mechanism of intact rock submitted to compressive stress.

(b) Failure mechanism of intact rock submitted to tensile stress.

Figure 3.8 Intact rock mass failure mechanisms (after Potvin, 1988a).



(a) Failure mechanism of a discrete block in an isotropic rock mass submitted to compressive stress

(b) Failure mechanism of a discrete block in an isotropic rock mass submitted to tensile stress.

(c) Failure mechanism of a discrete block in an anisotropic rock mass (elongated blocks parallel to stope surface) submitted to compressive stress.

(d) Failure mechanism of a discrete block in an anisotropic rock mass (elongated blocks parallel to stope surface) submitted to tensile stress.

(e) Failure mechanism of a discrete block in an anisotropic rock mass (elongated blocks perpendicular to stope surface) submitted to compressive stress.

(f) Failure mechanism of a discrete block in an anisotropic rock mass (elongated blocks perpendicular to stope surface) submitted to tensile stress.

Figure 3.9 Discrete rock mass failure mechanisms (after Potvin, 1988a).

are in a state of relaxation through gravity falling and slabbing (Figure 3.9*b*). For anisotropic rock masses the relative orientation of blocks to the opening surface influences the stability. For elongated blocks sub-parallel to a stope face, compressive stresses can lead to buckling failure (Figure 3.9*c*), whereas relaxation can lead to gravity falls (Figure 3.9*d*). For elongated blocks that are sub-perpendicular to a stope face, compressive stresses tend to stabilise the blocks due to a clamping action (Figure 3.9*e*), whilst in areas of relaxation, failure is a matter of kinematics and failure is due to gravity falling and sliding of rock blocks (Figure 3.9*f*).

Jointed Rock Behaviour

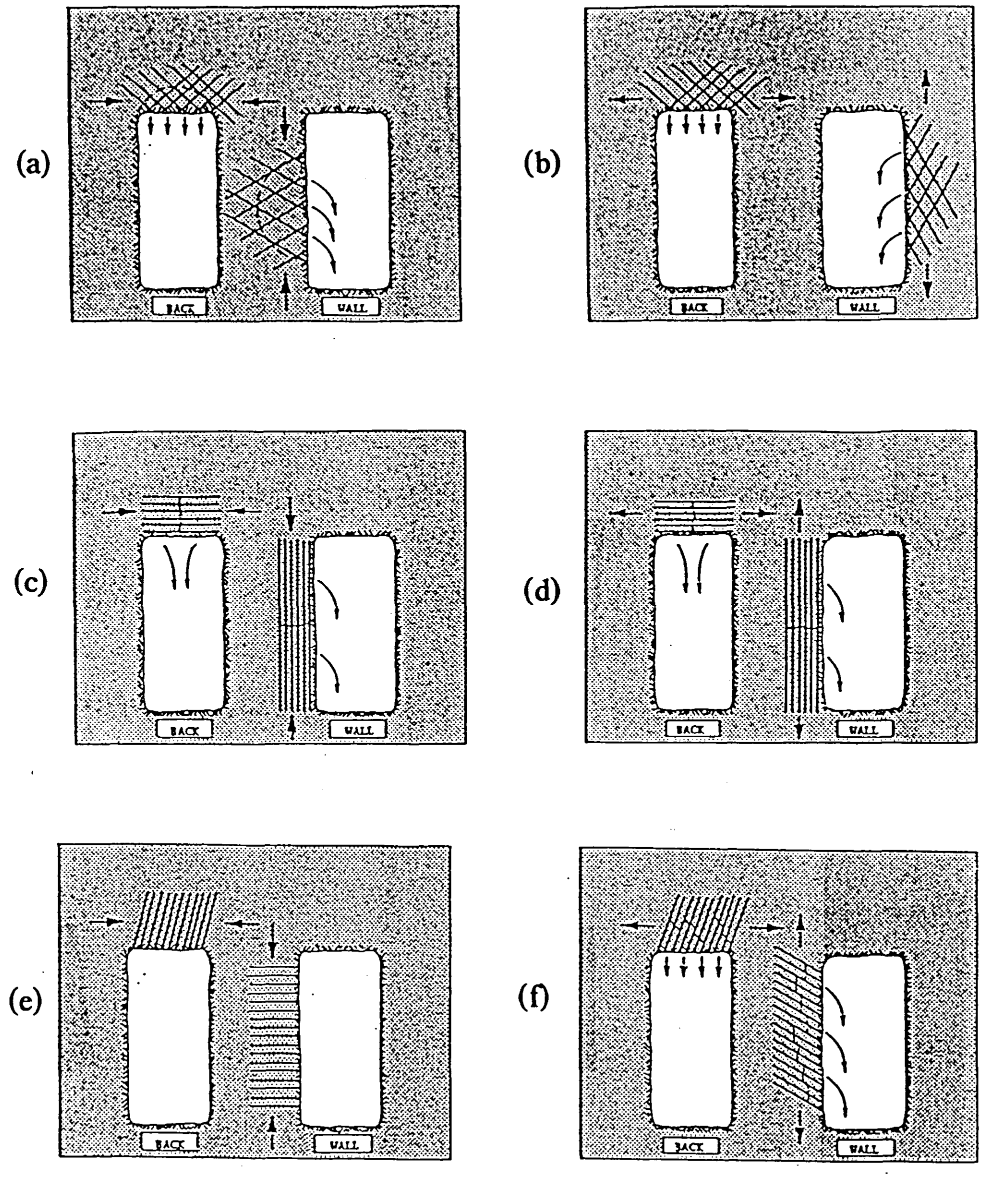
A jointed rock mass is defined as less competent ground, characterised by a high frequency of jointing, producing a rock mass consisting of small blocks. A jointed rock mass usually contains three or more well defined joint sets.

Failure of a jointed rock mass, whether in compression or relaxation, results in ravelling of blocks. Ravelling stops when a stable arch is formed by the interlocking of the surrounding rocks. When the critical stope span is exceeded, however, ravelling continues until better ground conditions are met or until the ground surface is met. Figure 3.10 shows the failure mechanisms exhibited by jointed rock masses.

3.3.5 Conclusions

The discussion above has highlighted the need to understand the nature of the rock mass if it is to be modelled properly. First, sufficient data must be collected in order to determine whether the rock mass exhibits intact, discrete or jointed rock behaviour. The size and shape of blocks must be determined in order to decide whether the rock mass can be considered as a continuum or discontinuum, whether it is isotropic or anisotropic, and whether it can be assumed to be linearly elastic or otherwise.

Once such criteria have been established a conceptual model of the rock mass can be developed, and to this model an appropriate modelling approach can be associated. There are many modelling approaches for open stope design and pillar design, and they are outlined in Section 3.4 and Section 4.3, respectively. However, as was stated in Section 3.3.2, rock mechanics is an engineering discipline where data are generally very limited and where advanced computational techniques are



(a) Failure mechanism of jointed rock in an isotropic rock mass submitted to compressive stress.

(b) Failure mechanism of jointed rock in an isotropic rock mass submitted to tensile stress.

(c) Failure mechanism of jointed rock in an anisotropic rock mass (elongated blocks parallel to stope surface) submitted to compressive stress.

(d) Failure mechanism of jointed rock in an anisotropic rock mass (elongated blocks parallel to stope surface) submitted to tensile stress.

(e) Failure mechanism of jointed rock in an anisotropic rock mass (elongated blocks perpendicular to stope surface) submitted to compressive stress.

(f) Failure mechanism of jointed rock in an anisotropic rock mass (elongated blocks perpendicular to stope surface) submitted to tensile stress.

Figure 3.10 Jointed rock mass failure mechanisms (after Potvin, 1988a).

normally inappropriate. This is especially the case in mine design, where the analysis should always take into account the quality and quantity of the data that are required for analysis, *e.g.*, intact rock properties, discontinuity data, *in situ* stress data.

Finally, modelling should never be perceived as a one run exercise. It is essential that the modelling should be carried out for ranges of plausible parameter values. Where the model output is relatively insensitive to parameter values then there is little need to worry; but where it is sensitive, the form of the sensitivity should suggest limits to the parameter values, or field experiments that ought to be performed. In many cases it is in the sensitivity of the results, to changes in parameter values and assumptions, that the model is most informing.

3.4 Approaches to Open Stope Design

Rock mechanics input in open stope design is related to:

- dimensioning of stopes and pillars
- layout of stopes and pillars within the orebody
- sequencing the mining of the orebody.

Fairhurst and Brady (1983) identified three possible approaches to underground excavation design, reflecting various levels of reliance on the principles of engineering mechanics. These approaches were termed extrapolating precedent practice, simple conceptual modelling, and advanced computer analysis, and they are critically examined below.

3.4.1 Extrapolating Precedent Practice

Historically, engineering design in rock mechanics has been based on past experience. Rock classification schemes were devised as a means of quantifying the often subjective parameters which describe the properties of a rock mass. Classification schemes take into account factors that are deemed to influence rock mass behaviour, and these can be divided into two broad categories:

- the *in situ* loading conditions
- the rock mass properties.

The *in situ* loading conditions are governed by the *in situ* stresses present at the site, as well as the size and shape of any openings, which alter this *in situ* stress field. The rock mass properties are what rock mass classification systems are most concerned with. Rock mass classifications attempt to quantify aspects of the rock mass in order to give a number that reflects the quality of the rock. The parameters used in the classification are either directly measured or scaled against descriptive scales, and can be divided into two categories:

- intact rock properties

- discontinuity properties.

The properties of intact rock include the uniaxial compressive strength, UCS, Young's modulus, E, and Poisson's ratio, ν. The properties of the discontinuities which are important to the behaviour of a rock mass were defined by Barton (1981) as orientation, spacing, persistence, roughness, wall strength, aperture, filling, seepage, number of sets of discontinuities, block size. For classification purposes these parameters are grouped into more general categories:

- size, shape and orientation of blocks

- friction angle of joints

- joint stiffness.

Rock Mass Classification Systems

Terzaghi (1946) developed one of the first rock mass classification systems, in order to estimate the loads to be supported by steel arches in tunnels. Terzaghi used subjective descriptions of the rock mass such as intact, stratified, blocky and seamy, crushed *etc.*, to indicate the degree of fracturing. Linked to each rock description was an estimated rock load and probable support requirement.

Deere (1964) developed a rock mass classification system to quantify the competence of rock obtained from drill core. The rock quality designation, RQD, is defined as the total length of core, greater than 10 cm in length, divided by the total length of core. RQD is used as the basis for the selection of tunnel support, but it is most widely used as a parameter in other rock mass classification systems. This is because of the ease of its measurement and the fact that it gives some indication about the degree of fracturing present in the rock mass.

Barton *et al.* (1974) developed the NGI rock mass classification system for tunnel design. The rock mass classification system quantifies factors which affect rock mass stability, such as block size, discontinuity friction and stress effects, to produce a rock quality index, Q. The NGI rock mass classification has been used widely in tunnelling work but is not commonly used in underground mining. This is because the NGI rock mass classification contains a design element, in the form of the stress reduction factor, which relates only to tunnelling. The rock mass classification part of the NGI system, however, has been used in mining, by Mathews *et al.* (1981), as part of a general empirical open stope design method. Tables 3.1 and 3.2 show the NGI rock mass classification system.

Bieniawski (1976) developed the CSIR rock mass classification system for estimating the support requirements for tunnelling. The rock mass classification system derives a rating between 0 and 100, called the Rock Mass Rating, RMR, by adding together the five components that make up the rating. The rating given to each individual component has altered since its introduction, but the values given in Hoek and Brown (1980) are the most widely used (Table 3.3).

The NGI and CSIR systems provide an objective means of using past experience in the planning of new excavations in a variety of rock environments. However, as has been stated above, they were developed based on tunnelling case histories, and their design charts relating rock mass classification rating with span of openings, support requirements and stand up times, are only applicable to tunnels. Laubscher (1977, 1984) modified the CSIR rock mass classification system to produce the mining rock mass rating (MRMR). The MRMR system also contains a design element to take account of loading effects and design constraints of the engineering structure. Laubscher's system has been used successfully to predict stable spans, stand up times, cavability of ore and angle break in hangingwall caving.

Despite the emphasis that all these rock mass classification systems place on the importance of discontinuities and rock mass quality, they do not take account of the mechanics of a problem. The method of allowing for changes in stress due to an excavation, by the use of an adjustment factor is unsuitable. The large possible range for this parameter means that the overall rock mass classification is highly sensitive to the values assigned to it.

Description	*Value*	*Notes*
1. ROCK QUALITY DESIGNATION	RQD	
A. Very poor	0 - 25	1. Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.
B. Poor	25 - 50	
C. Fair	50 - 75	
D. Good	75 - 90	2. RQD intervals of 5, i.e. 100, 95, 90 etc are sufficiently accurate.
E. Excellent	90 - 100	
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use $(3.0 \times J_n)$
G. Three joint sets plus random	12	2. For portals use $(2.0 \times J_n)$
H. Four or more joint sets, random, heavily jointed 'sugar cube', etc	15	
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact and		
b. Rock wall contact before 10 cms shear.		
A. Discontinuous joints	4	
B. Rough or irregular, undulating	3	
C. Smooth, undulating	2	
D. Slickensided, undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.
E. Rough or irregular, planar	1.5	
F. Smooth, planar	1.0	2. J_r = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are orientated for minimum strength.
G. Slickensided, planar	0.5	
c. No rock wall contact when sheared.		
H. Zone containing clay minerals thick enough to prevent rock wall contact.	1.0	
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact.	1.0	

4. JOINT ALTERATION NUMBER	J_a	ϕ_r(approx.)	
a. Rock wall contact.			
A. Tightly healed, hard, non-softening, impermeable filling	0.75		
	J_a	ϕ_r(approx.)	
B. Unaltered joint walls, surface staining only	1.0	(25° - 35°)	
C. Slightly altered joint walls non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0	(25° - 30°)	1. Values of ϕ_r, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20° - 25°)	
E. Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2mm or less in thickness)	4.0	(8° - 16°)	
b. Rock wall contact before 10 cms shear.			
F. Sandy particles, clay-free disintegrated rock etc	4.0	(25° - 30°)	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5mm thick)	6.0	(16° - 24°)	
H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous, < 5mm thick)	8.0	(12° - 16°)	
J. Swelling clay fillings, i.e. montmorillonite (continuous, < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water	8.0 - 12.0	(6° - 12°)	
c. No rock wall contact when sheared.			
K. Zones or bands of disintegrated L. or crushed rock and clay (see M. G,H and J for clay conditions)	6.0 8.0 8.0 - 12.0	(6° - 24°)	
N. Zones or bands of silty- or sandy clay, small clay fraction, (non-softening)	5.0		
Q. Thick, continuous zones or P. bands of clay (see G, H and R. J for clay conditions)	10.0 - 13.0 13.0 - 20.0	(6° - 24°)	

Table 3.1 The NGI rock mass classification scheme – definition of parameters RQD, J_n, J_r and J_a (after Hoek and Brown, 1980).

5. JOINT WATER REDUCTION FACTOR	J_w	approx. water pressure (Kgf/cm²)	
A. Dry excavations or minor inflow, i.e. < 5 lit/min. locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
D. Large inflow or high pressure, considerable outwash of fillings	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure continuing without decay	0.1 - 0.05	> 10	

6. STRESS REDUCTION FACTOR

a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.

	SRF	
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10.0	1. Reduce these values of SRF by 25 - 50% if the relevent shear zones only influence but do not intersect the excavation.
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50m)	5.0	
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50m)	2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5	
E. Single shear zones in competent rock (clay free), (depth of excavation < 50m)	5.0	
F. Single shear zones in competent rock (clay free), (depth of excavation > 50m)	2.5	
G. Loose open joints, heavily jointed or 'sugar cube' (any depth)	5.0	

b. Competent rock, rock stress problems

	σ_c/σ_1	σ_t/σ_1	SRF
H. Low stress, near surface	>200	>13	2.5
J. Medium stress	200-10	13-0.66	1.0
K. High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10-5	0.66-0.33	0.5-2
L. Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10
M. Heavy rock burst (massive rock)	<2.5	<0.16	10-20

2. For strongly anisotropic virgin stress field (if measured) : when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.

3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

c. Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure

	SRF
N. Mild squeezing rock pressure	5-10
O. Heavy squeezing rock pressure	10-20

d. Swelling rock, chemical swelling activity depending upon presence of water

	SRF
P. Mild swelling rock pressure	5-10
R. Heavy swelling rock pressure	10-20

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in the tables:

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay free rock masses : RQD = 115 - 3.3J_v (approx.) where J_v = total number of joints per m³ (RQD = 100 for J_v < 4.5)
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or only occasional breaks in the core due to these features, then it will be more appropriate to count them as "random joints" when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the *weakest significant joint set or clay filled discontinuity* in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q . *The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.*
4. When a rock mass contains clay, the factor SRF appropriate to *loosening loads* should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to present or future in situ conditions. A very conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Table 3.2 The NGI rock mass classification scheme – definition of parameters J_w and SRF (after Hoek and Brown, 1980).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>8 MPa	4-8 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial compressive strength	>200 MPa	100-200 MPa	50-100 MPa	25-50 MPa	10-25 MPa	3-10 MPa	1-3 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90%-100%	75%-90%	50%-75%	25%-50%	<25%		
	Rating		20	17	13	8	3		
3	Spacing of joints		>3m	1-3m	0.3-1m	50-300mm	<50mm		
	Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1 mm Hard joint wall rock	Slightly rough surfaces Separation <1 mm Soft joint wall rock	Slickensided surfaces or Gouge <5 mm thick or Joints open 1-5mm Continuous joints	Soft gouge >5mm thick or Joints open >5mm Continuous joints		
	Rating		25	20	12	6	0		
5	Ground water	Inflow per 10m tunnel length	None		<25 litres/min.	25-125 litres/min.	>125 litres/min.		
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	OR 0		OR 0.0-0.2	OR 0.2-0.5	OR >0.5		
		General conditions	OR Completely dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Rating		10		7	4	0		

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100—81	80—61	60—41	40—21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 5m span	6 months for 4 m span	1 week for 3 m span	5 hours for 1.5 m span	10 min. for 0.5m span
Cohesion of the rock mass	>300 kPa	200-300 kPa	150-200 kPa	100-150 kPa	<100 kPa
Friction angle of the rock mass	>45°	40°-45°	35°-40°	30°-35°	<30°

TABLE 6 - THE EFFECT OF JOINT STRIKE AND DIP ORIENTATIONS IN TUNNELLING

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0°-20° irrespective of strike
Drive with dip		Drive against dip				
Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Unfavourable

Table 3.3 Definition of parameters in the CSIR rock mass classification scheme (after Hoek and Brown, 1980).

Rock mass classification systems are useful, however, when they are just used to give a quality rating to the rock mass. Pakalnis *et al.* (1985) developed a geonumerical model for use in open stope design. They attempted to empirically relate optimum stope geometries to the rock quality and the stress configuration that surrounded individual stopes. The study allowed mining engineers to predict the volume of ore, with a particular RMR value, that could be excavated to give a certain cumulative dilution. They found that dilution was not particularly sensitive to the stope configuration for the isolated case and hence no optimum stope configuration could be determined. The study was also unable to say anything about the effect that stress distributions around two or more adjacent stopes would have on dilution levels.

Stability Graph Method

Mathews *et al.* (1981) developed an empirical relation between rock mass rating, mining depth and open stope dimensions. Their initial study was of open stopes in three Canadian mines but it has subsequently been investigated by Canadian universities and extended by Potvin *et al.* (1987) to produce the *stability graph method* for open stope design. The design procedure is able to take account of both stress controlled and structurally controlled failure and so can be seen as a simple means of giving initial design estimates for open stope dimensions, and also highlights areas which may require further, more detailed analysis.

The stability graph method studies the stability of an open stope, plane by plane. For each of the planes (roof, hangingwall, footwall, stope ends), the stability can be represented by a stability number. The stability number compiles the effect of the principal geotechnical factors influencing the stability of open stopes. Mathews *et al.* (1981) used the hydraulic radius to account for the size and shape of the plane being investigated. The hydraulic radius, S, is defined as the ratio of the surface area to the perimeter of the particular plane.

A relationship between the stability number and the hydraulic radius was derived by comparing them on a semi-log graph (Figure 3.11). The stability of the plane being investigated can be assessed from the graph according to where it plots with respect to the three zones: stable, potentially unstable, and potentially caving.

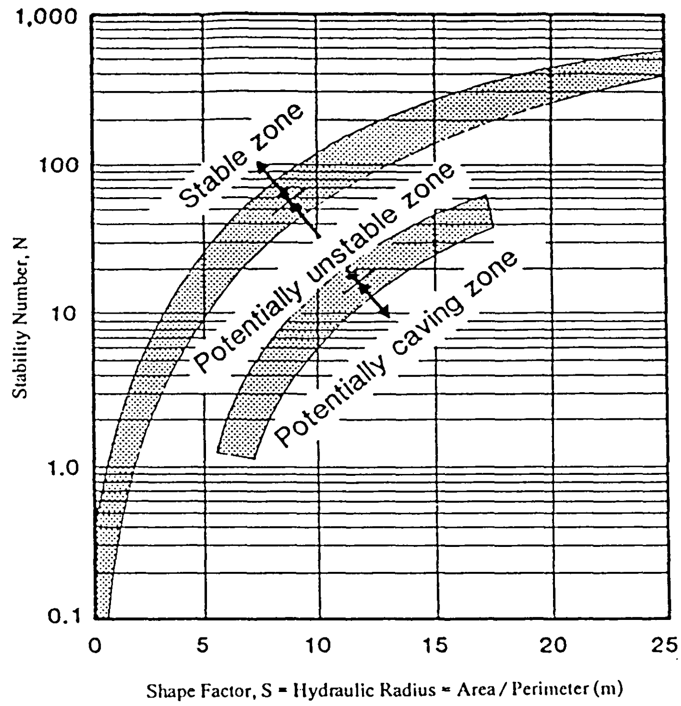
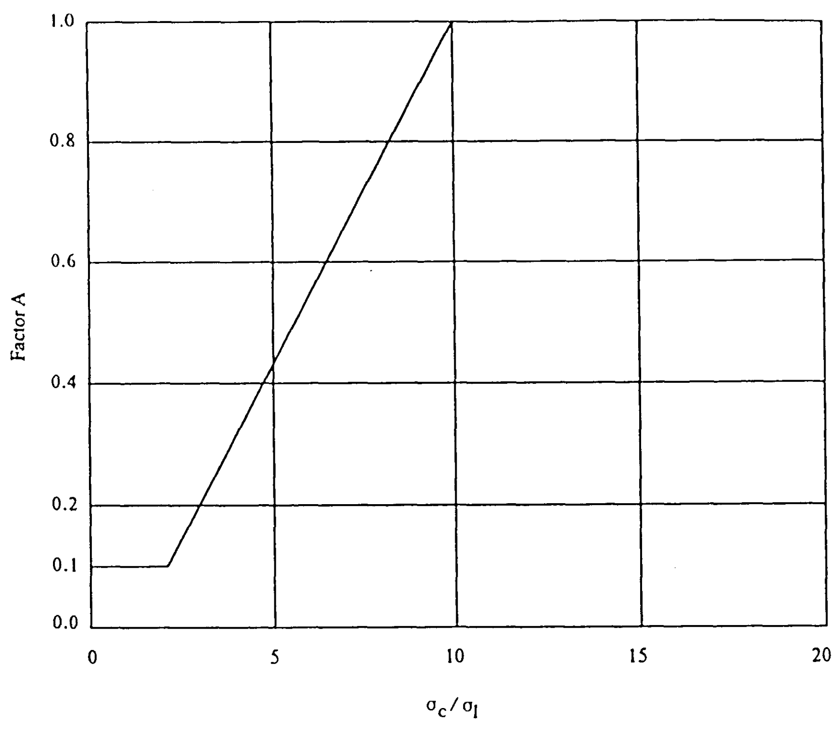


Figure 3.11 The stability graph.



σ_c = Uniaxial compressive strength of intact rock

σ_I = Induced compressive stress

Factor A = 1 if $\sigma_I < 0$

Figure 3.12 Graph of the effect of stress – Factor A.

The stability number, N, can be calculated using the formula:

$$N = Q' \times A \times B \times C \tag{3.1}$$

where Q' is a modified NGI rock mass quality index. It is based on the first two quotients of the NGI classification.

$$Q' = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \tag{3.2}$$

RQD	=	rock quality designation – the percentage of rock in a core which is greater than 10 cm in length.
J_n	=	joint set number – assesses the effect of the number of joint sets in the rock mass.
J_r	=	joint roughness number – characterises the shape and irregularities of fracture surfaces.
J_a	=	joint alteration number - gives an approximation of the friction angle of the joint surface. It considers the presence of infilling and the condition of the joint surfaces.

The third quotient in the NGI classification is eliminated because the stress reduction factor does not represent open stopes well.

Factor A accounts for the rock stress. The rock stress factor replaces the stress reduction factor to more accurately reflect the stresses acting on exposed surfaces of open stopes. Factor A is a function of the ratio of intact rock strength to induced stress where :-

- intact rock strength is the uniaxial compressive strength of the rock.
- induced stress is defined as the induced stress acting parallel to the exposed stope wall or roof under analysis.

The uniaxial compressive strength can be determined through laboratory testing, but the only technique for estimating the stresses induced parallel to the stope planes is numerical modelling. Factor A can then be determined using Figure 3.12. As can be seen from Figure 3.12, the minimum value for A has been set to 0.1 for the case where the ratio of the uniaxial compressive strength of the rock to the induced stress is less than 2. The reason for this is that case histories have shown that when the shape factor is small, the stability problems due to compressive failure are minor. If the induced stresses are negative (*i.e.*, tensile) then factor A is set to 1.

Factor B is the rock defect orientation factor. This factor is included to account for the presence of persistent structures intersecting exposed surfaces. The value of B depends on the relative orientation of these structures with the investigated plane (Figure 3.13). Factor B takes into account not only discontinuities that strike parallel to the stope surface, but also those that strike at any orientation. Kinematically, however, the effect of discontinuities on stope stability diminishes as the difference between the strike of the surface and the strike of the joint set goes from parallel to perpendicular, as is represented in Figure 3.13. The joint set used to calculate factor B is the one that is most likely to initiate failure. The engineers at a mine site would know from experience and visual observations which set this is. If there is no past experience then the most prominent joint set can be used, or alternatively kinematic analysis as described in Priest (1985) can be used to identify the joint set.

Factor C is the orientation design surface factor. Stope backs or roofs are inherently less stable than walls because of the influence of gravity. Factor C is given in Figure 3.14.

The shape factor, S, or hydraulic radius of a plane is defined as the ratio of the surface area to the perimeter, and so takes into account the effect of the size and shape of the stope surfaces.

$$Hydraulic\ Radius, S = \frac{surface\ area}{perimeter} \tag{3.3}$$

As the length to width ratio of a plane increases beyond $4:1$ the hydraulic radius remains relatively constant. This corresponds to the best shape (long and narrow) for a plane of a given area.

The stability graph method described above is for unsupported open stopes. Mechanical stope support in the form of rock and cable bolting is widely used. Potvin *et al.* (1987) attempted to account for such support by modifying the

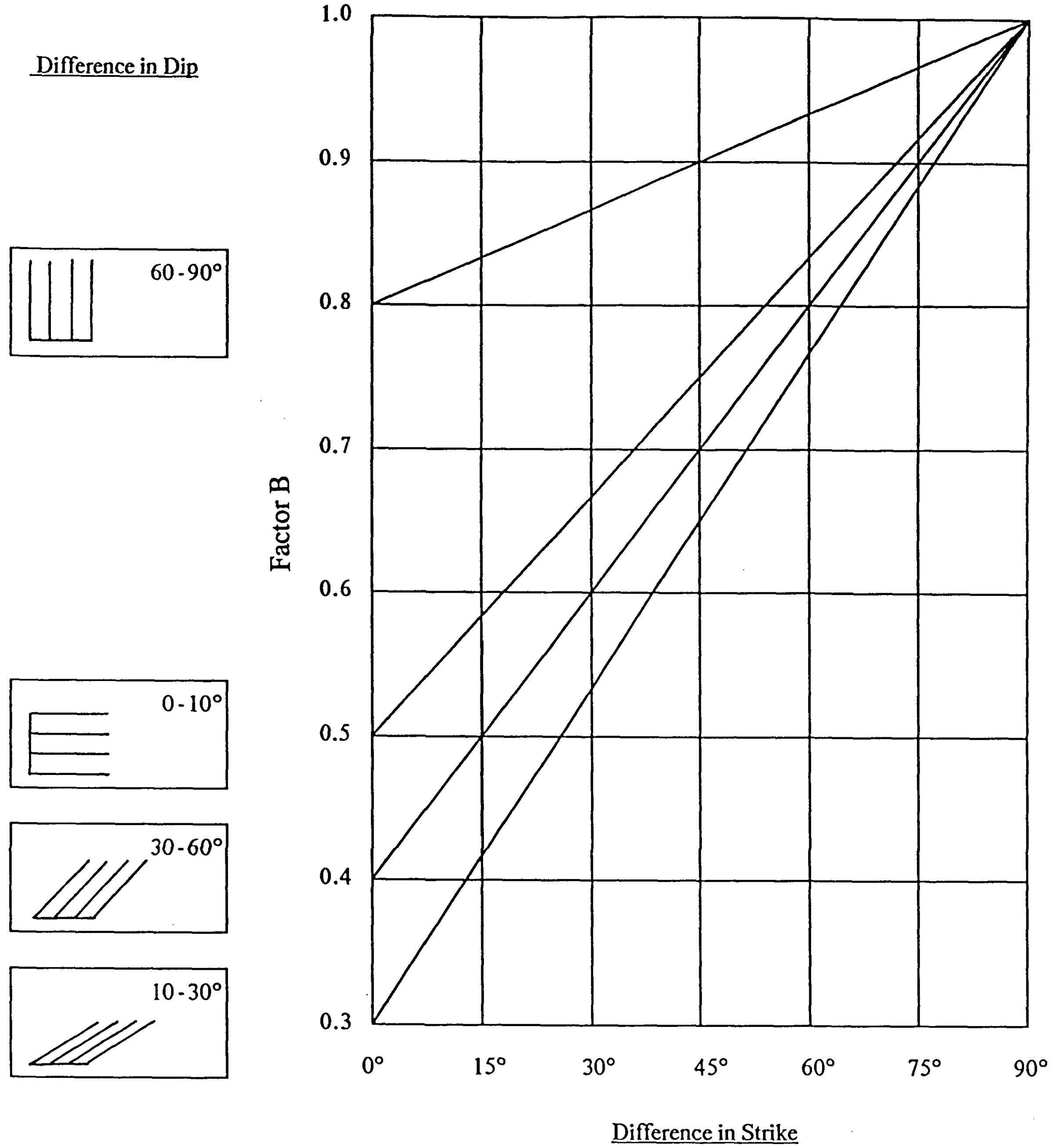
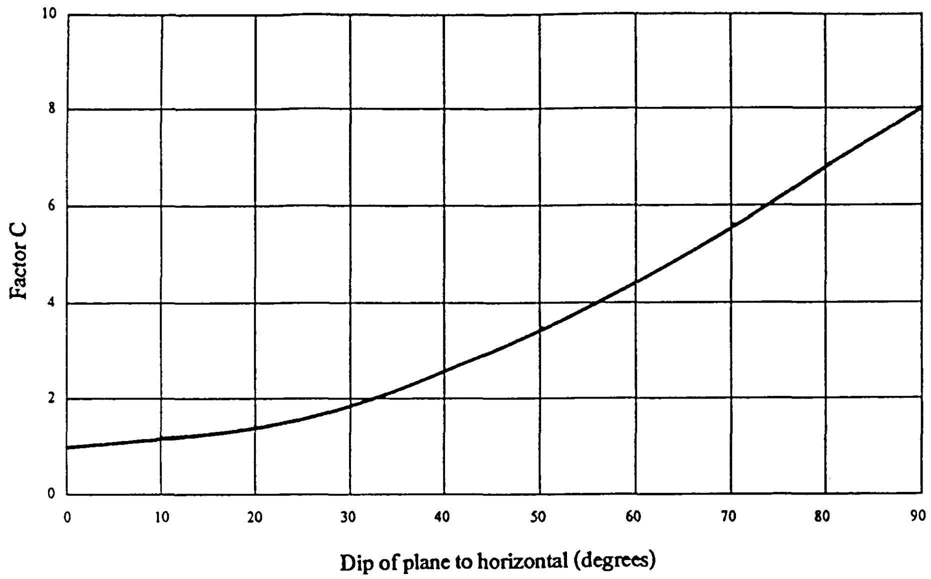
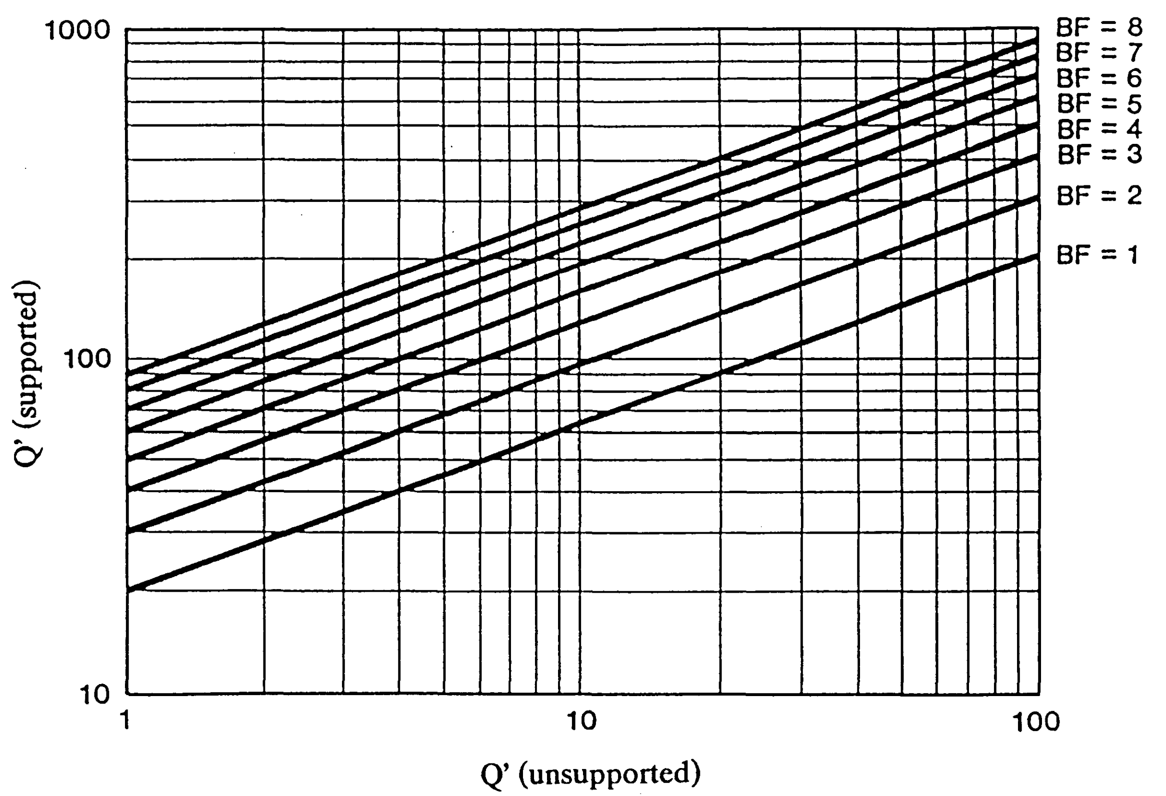


Figure 3.13 Graph of the effect of rock defect orientation – Factor B.



[Factor C = 8 - 7 cos (angle of dip)]

Figure 3.14 Graph of the effect of the dip of the stope plane – Factor C.



[Bolting Factor (BF) = metres of cable / m^2]

Figure 3.15 Graph of the empirical relation between density of cable bolting and improvement in rock mass quality (after Bawden, 1988b).

factor Q' (Equation 3.2) in order to take account of the level of support used. They devised a bolting factor, which was defined as:

$$Bolting\ Factor = \frac{number\ of\ bolts}{m^2} \times bolt\ length \tag{3.4}$$

Potvin *et al.* (1987) only considered full friction support, such as cable bolts, rebar and Swellex, for the calculation of the bolting factor. Potvin *et al.* (1987) produced a chart of $Q'supported$ against $Q'unsupported$. $Q'unsupported$ was the original Q' from equation 3.2 (Figure 3.15).

Potvin *et al.* (1987) had a database of 44 supported stopes, and they showed that many stopes that had previously been predicted unstable, but which were in fact stable, plotted in the stable region with the modified stability number due to support. There were some flaws in the database, but most of these were accounted for by poor installation of the cable bolting. This method of accounting for support shows promise for use by mine operators, as the stability graph method itself only gives subjective descriptions of rock mass behaviour – stable, potentially unstable, potentially caving.

Another area that is being investigated is the linking of stope dilution to the stability graph. Mine operators can already do this if they have a sufficient enough database of stopes with accurate records of their average dilution. Dilution percentage lines plot parallel to the lines dividing the stability graph into zones, which is logical since the zones indicate increasing instability, which is one of the causes of dilution. One problem with this concept is the effect that blasting has on dilution. Where blasting is of a high quality, dilution is due mainly to stope plane instability, but poor blasting can lead to high dilution, even where a stope is predicted to be relatively stable.

3.4.2 Simple Conceptual Modelling

Simple conceptual modelling is increasingly being used in underground excavation design. The methodology utilises quite simple numerical techniques, but its strength comes from the fact that it was designed specifically for rock mechanics, and has not been imported from another branch of engineering. Simple conceptual modelling has evolved recognising the basic problems that arise in excavation design, especially stope design. These problems include the difficulty in obtaining *in situ* rock mass properties, the *in situ* state of stress, and the location and orientation of discontinuities within the rock mass. The quality of the data available for design is often poor, and the design methodology must be flexible enough to take this into account.

From the rock mass characterisation data, a simple conceptual model of the rock mass, incorporating the main mechanical features of the rock mass, can be developed. Section 3.3.3 reviewed the role of conceptual models in rock mechanics and Chapter 6 develops some simple models for open stope design. The next stage is the design analysis, using a relatively simple computational method, which allows for the key modes of rock mass response. Simple constitutive behaviour of the rock mass is usually assumed, *e.g.*, linear elasticity. There is no need for more complex constitutive behaviour, since the initial problem is to recognise defects or limitations in the design. The approach adopted after the initial design is one of an observational methodology. The essential component of an observational methodology is the monitoring of the rock mass, followed by retrospective analysis of the measured *in situ* rock mass response. Figure 3.16 gives the components and logic of the methodology.

In numerical analysis of open stope design, three general approaches have been identified from the literature.

Stress Simulation

Numerical models are useful in parameter studies for comparing the effect that changes in input parameters have on output. This is a means of determining the most critical variables in the design, so as to concentrate on the accurate modelling of these in later, more detailed design. Parameter studies are also useful as an aid in the selection of optimum stope size, shape and extraction sequence. The absolute values of stress magnitudes obtained from the numerical analysis should be used with caution.

Starfield and Crouch (1973) used a three-dimensional displacement discontinuity program to demonstrate the use of numerical analysis in mine planning. They modelled two possible layouts in a steeply dipping, tabular orebody – one layout with rib pillars between stopes, the other with crown pillars. The results showed that the two layouts gave considerably different stress distributions. Chen *et al.* (1983) used a two-dimensional non-linear finite element program for the

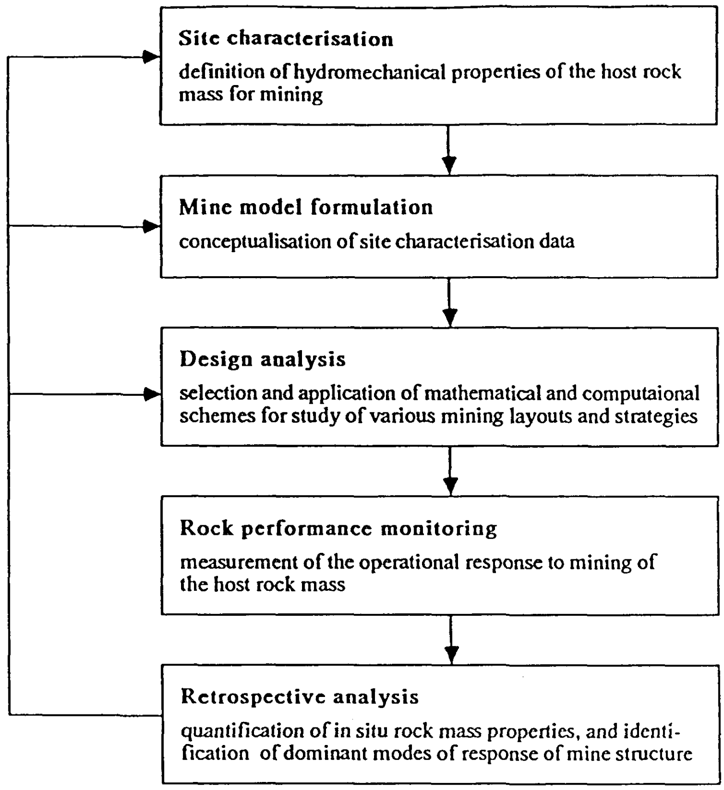


Figure 3.16 Components and logic of a rock mechanics program (after Brady and Brown, 1985).

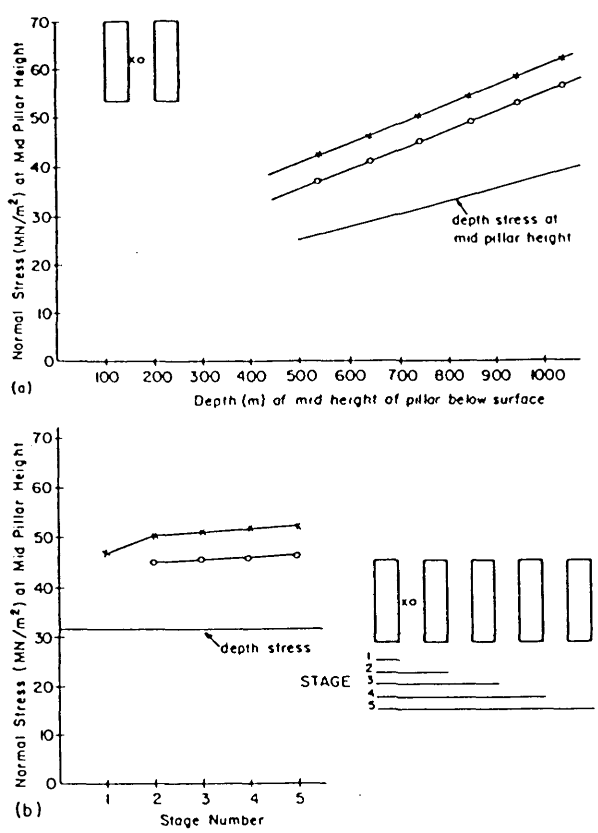


Figure 3.17 Parameter study of open stope geometries and mining sequence at the No.7 orebody, Mount Isa mine, Australia (after Bywater *et al.*, 1983).

dimensioning of open stopes. They investigated stope span and rib pillar width, influence of the number of open stopes on boundary stress distributions, influence of stope height and effect of delayed backfilling. Bywater *et al.* (1983) used the three-dimensional displacement discontinuity program, NFOLD, to conduct parameter studies in the narrow, steeply dipping, tabular lead/zinc orebodies at Mount Isa mine, Australia. Figure 3.17 shows the results from the modelling of various geometries and mining sequences.

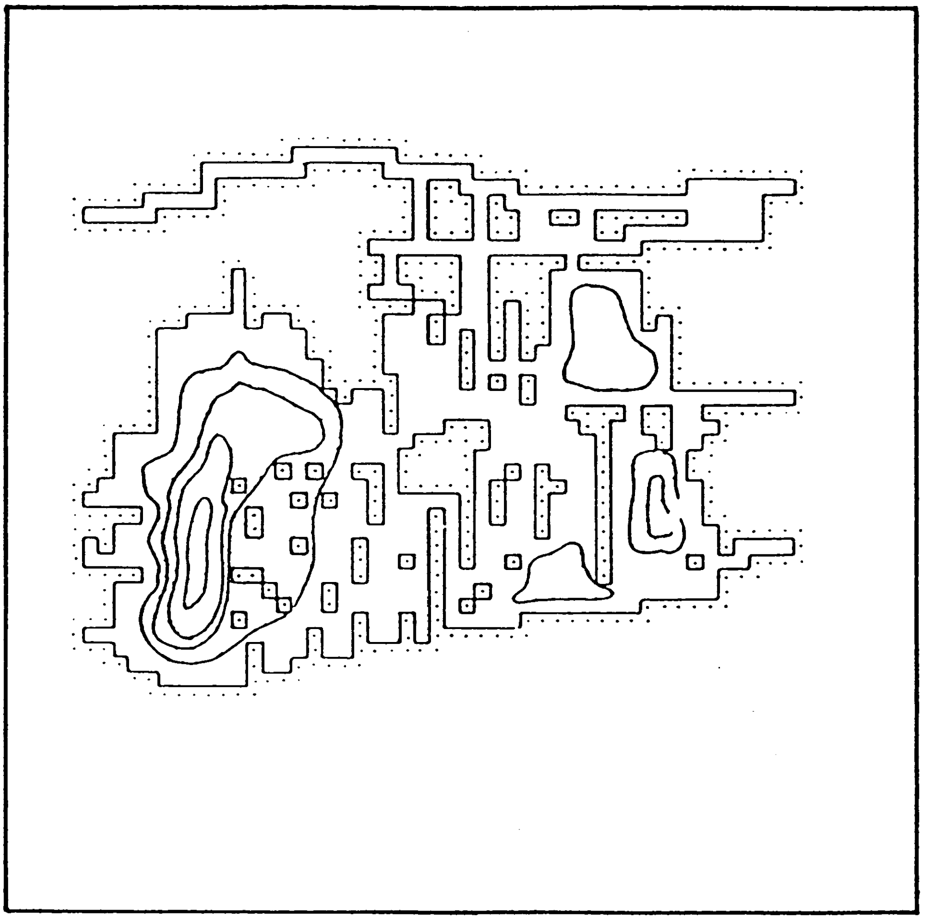
Rock Mass Deformation Simulation

By assigning elastic or plastic properties to the rock mass, the theoretical displacement distributions and induced stresses can be calculated around an excavation. Using back analysis of measured displacements or stresses the numerical model can be calibrated and used in subsequent design. However, care must be taken with the calibration because if structure plays an important role in the deformation, it may not have been taken into account by the numerical modelling. Consequently, measured displacements would not relate to the calculated displacements.

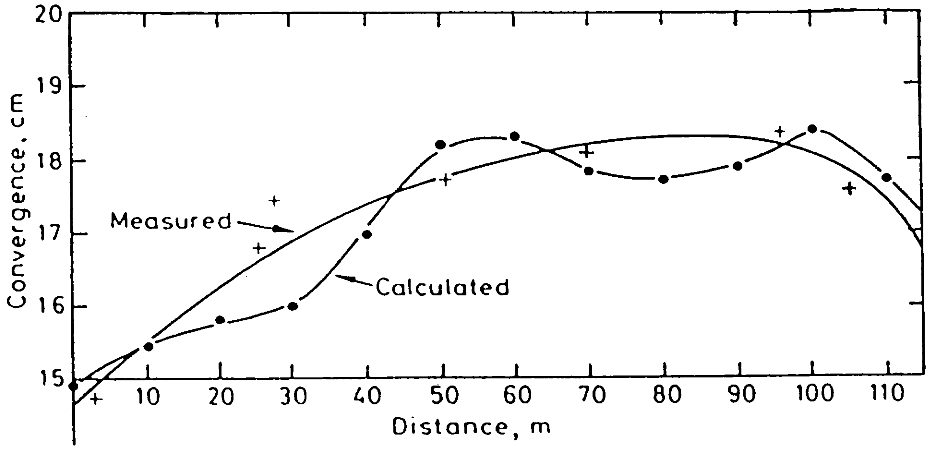
Pariseau *et al.* (1985) used a finite element program to model stope and pillar layouts at Carr Fork mine, Utah and Homestake mine, South Dakota. They compared the computed displacements with displacements measured using multi-anchor borehole extensometers. They found that two scale factors, one for elastic moduli, and one for strengths, were sufficient to calibrate the finite element model of the stope layouts to the observed behaviour. This calibrated model was then used with the accumulated rock mass data to carry out detailed analysis of production layouts and mining sequences.

Batchelor and Kelly (1975) used a three-dimensional displacement discontinuity program to model stopes at Wheal Jane mine, Cornwall. They varied the Young's modulus of the surrounding rock mass and the orebody until computed stope closure matched measured stope closure (Figure 3.18). Bywater *et al.* (1983) used absolute stress measurements in the rock mass to back analyse against computed stresses. They also used visual observations of failure to link predicted stress levels with degree of failure. The calibrated model was then used for optimising stope layouts and sequences. Such analysis is being adopted by many mines (Hammett and McKervey, 1985; Bawden and Milne, 1987; Esterhuizen, 1987).

(a)



(b)



(a) Convergence contours of existing stopes as calculated by program

(b) Comparison of calculated and measured convergence on section through 333 stope.

Figure 3.18 Back analysis of measured convergence and computed convergence of the 333 stope, Wheal Jane mine, Cornwall (after Batchelor and Kelly, 1975).

Stress Controlled Failure Simulation

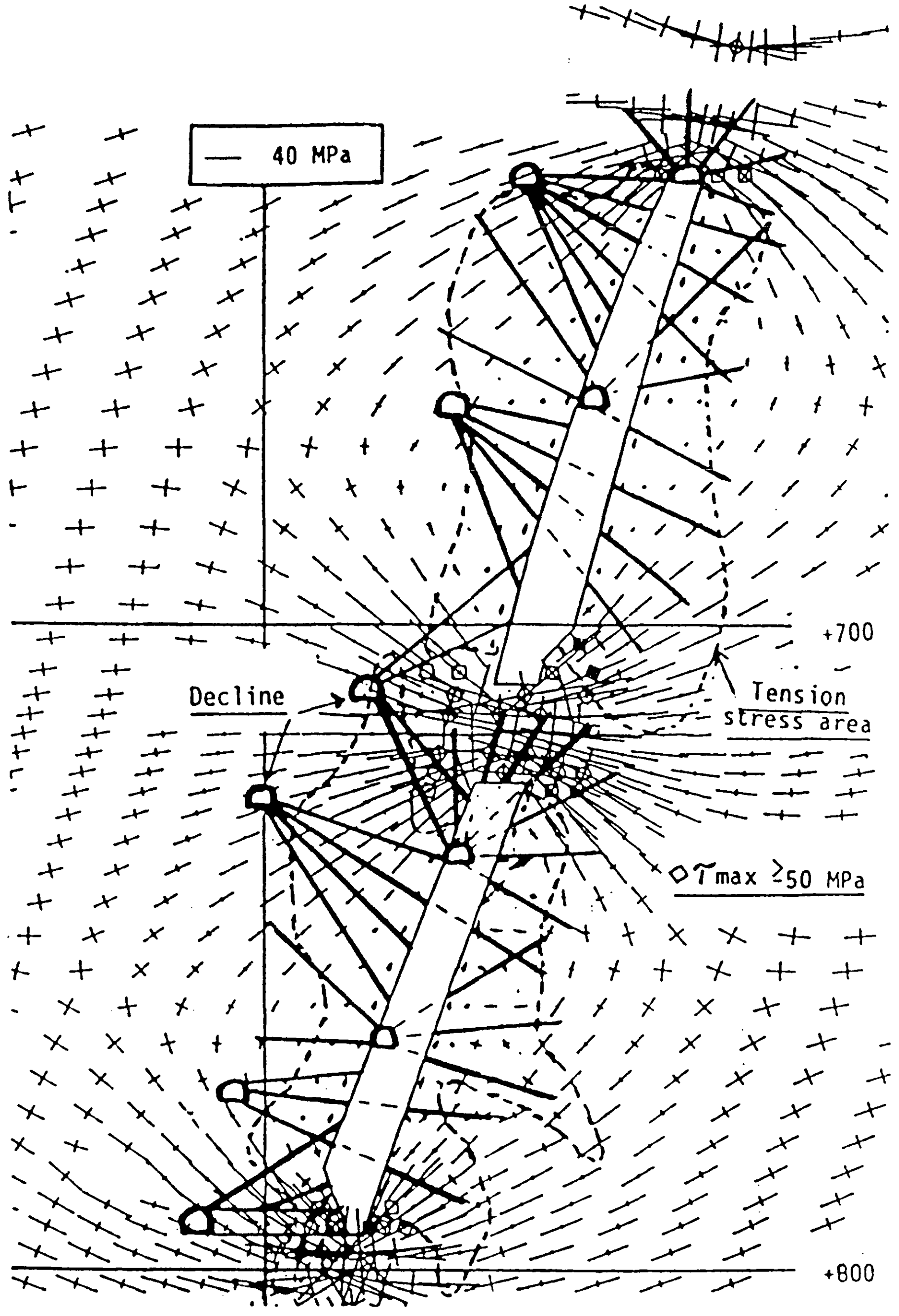
Using finite element or boundary element programs the induced stresses around excavations can be calculated. Zones of stress controlled failure can be identified by comparing the calculated induced stress in the vicinity of openings with a failure criterion for the rock mass. The Hoek-Brown failure criterion is the most widely accepted failure criterion in this type of analysis, but it has to be used with caution.

Lover *et al.* (1983) used two-dimensional boundary element and finite element programs for modelling open stope layouts at the Kotalahti mine, Finland. Using the output from the analyses, zones of tensile stress and zones where compressive stresses exceeded the rock mass strength were plotted. Areas were pin-pointed where tensile and shear stresses were most critical, in order that cable bolting could be installed in these areas prior to mining. The cable bolts were placed such that they were aligned perpendicular to the maximum compressive stress (Figure 3.19).

Mathews *et al.* (1983) used a simple two-dimensional boundary element program to investigate the effect of mining stopes close to one another. Zones of tensile stress were plotted as were areas where stress exceeded the rock mass strength. Yu and Quesnel (1984) and de Souza *et al.* (1986) both used the two-dimensional boundary element program BOUND with the Hoek-Brown failure criterion. High compressive stress concentrations were found to correspond to spalling, especially at sharp corners which should be avoided in stope design. Zones of predicted tensile stress in hangingwalls corresponded to minor sidewall caving of weak rocks or failure along discontinuities.

3.4.3 Advanced Computer Analysis

As Chapter 2 highlighted, there now exists a wide range of numerical analysis programs in engineering rock mechanics. Within the last ten years computer programs have been developed which incorporate more general constitutive laws for the rock mass, and which are able to model in three dimensions. The methodology used with this advanced computer analysis makes no attempt to identify potential failure mechanisms at the pre-analysis stage, and no conceptualisation of the rock mass is required. Rather, the response of the rock mass evolves in the numerical analysis due to the imposed boundary conditions and the constitutive



Stress tensors calculated using a finite element program.
In Situ stresses taken as σ_H = 50 MPa , σ_V = 22 MPa.
Areas with tensile stresses are contoured by dotted lines.

Figure 3.19 Pre-placed cable bolting plan, Kotalahti mine, Finland (after Lover *et al.*, 1983).

behaviour of the rock mass. Such analysis requires programs that can model all the complexities of a rock mass, and as such must be able to model non-linearities due to compressive or tensile fracture of intact rock, slip on discontinuities, and block rotation and translation.

Programs capable of this modelling exist in the form of distinct element and finite element programs. Nuclear waste repository design has adopted this advanced design methodology (Whyatt and Julien, 1988) and as part of the methodology the approach has been to try to gather sufficient data to completely describe the rock mass. In mining, little use has been made of such advanced analysis with exceptions being Pariseau *et al.* (1984) and Beer *et al.* (1983).

Brady and St. John (1982) questioned the use of these advanced programs. They argued that no modelling philosophy had been developed for their use, particularly those that incorporated non-linear constitutive behaviour or a coupled response. Brady and St. John (1982) also noted that little verification against physical models had been conducted for programs incorporating non-linear behaviour. This is still true today, and the methodology adopted for these advanced computer programs tends to be one designed for aeronautical or mechanical engineering problems, where all aspects of the material under investigation are fully understood. Since the rock mass can never truly be fully understood in terms of its geomechanical behaviour, and the boundary conditions such as *in situ* stresses are heterogeneous and can never be known with complete certainty, then this advanced computer analysis methodology is deemed inappropriate to mining.

A recent development in numerical analysis has been the coupling of existing programs, to produce hybrid programs (Section 2.5). These hybrid programs, particularly the distinct element - boundary element (Lorig and Brady, 1984) and the finite element - boundary element programs (Meek and Beer, 1984), offer the most potential for use in underground hard rock mine design. By coupling the boundary element with the distinct element or finite element method, the advantage of the boundary element method for modelling the far-field linear elastic behaviour is combined with the power of the other methods for modelling the near-field rock mass response. To date, the engineering value of such programs has yet to be realised, but in the future coupled programs will be increasingly used (Brown, 1987). In order that a particular modelling procedure is applied in the correct manner, a modelling philosophy has to be developed, that defines the criteria linking modelling procedures to design problems. Brady (1987) and Pan (1989) have produced tentative conceptual models for rock masses and linked to these the appropriate advanced analysis. Such work is at an early stage in its development, but it is clear that with the increasing power of micro-computers, and hence the increasing ease with which such programs can be run by engineers at mine sites, that the use of hybrid programs in mine design will become more widespread.

3.5 Adopted Approach to Open Stope Design

3.5.1 Introduction

In Section 3.4 the various design methodologies that have been used in practice were outlined. Until recently most mines have adopted the simplest of these design approaches, namely precedent practice or empirical methods. This was due to the fact that until recently no other approach was available, and then when analytical methods did become available, they were found to be of little use to mine operators. This was due to two main reasons:

- Mine operators wanted a predictive tool that could tell them what would happen to the stope in terms of support required and dilution expected. The analytical models were unable to give any direct answers to these questions.

- The models themselves required geotechnical information that very often was not available, and was expensive to measure.

Increasingly, mines collect geotechnical information such as the intact rock properties, discontinuity data, and *in situ* stress field present at the mine site. With this increase in geotechnical data, more accurate conceptual models of the rock mass can be developed, and hence a more detailed understanding of the behaviour of the rock mass and the likely failure mechanisms, can be established. With this increase in data, analytical and numerical models are more able to predict what would happen to the rock mass as a result of creating an excavation. In addition, modelling techniques have improved, such that models can say something about stope behaviour that a mine operator can act on. Hence, stope dimensioning, based on both the predicted stability of the stope and mining economics, became possible.

With the development of more advanced numerical methods, there has been a move to using these in stope design. However, the quality and level of data that these models require could not be justified by every mining operation. Hence, the design approach outlined in Section 3.4.2, design by simple conceptual modelling, is the adopted approach, in this thesis, for open stope design. The following section describes its implementation and Figure 3.20 gives a flowchart of the logic behind the methodology.

3.5.2 Advanced Stope Design Methodology

Site Characterisation

The developed stope design methodology is based firmly on the observational principle (Section 3.4.2). The first stage of the design is the development of a site characterisation model, using all the geotechnical data available. For this characterisation model the following data are required:

- *in situ* stress field prior to mining
- strength and deformation properties of all the lithological units of interest
- location, dip, dip direction and material properties of the major discontinuities present.

These data are updated as part of the methodology, as more of the rock mass is exposed during mining, and after the performance of the rock mass has been monitored.

The main difficulty involved in site characterisation is obtaining representative samples of the rock mass. Generally specimens are small in comparison to the design problem, so there is uncertainty in relating the geomechanical properties of relatively small, intact specimens, to the *in situ* rock mass. The intact material properties, measured in the laboratory, include the uniaxial compressive strength, tensile strength, Young's modulus and Poisson's ratio of all the major rock types present. Discontinuity data can be obtained simply at the mine, using scanline techniques (Brady and Brown, 1985).

In Situ Stress

The *in situ* stress field present prior to mining is difficult to measure or estimate.

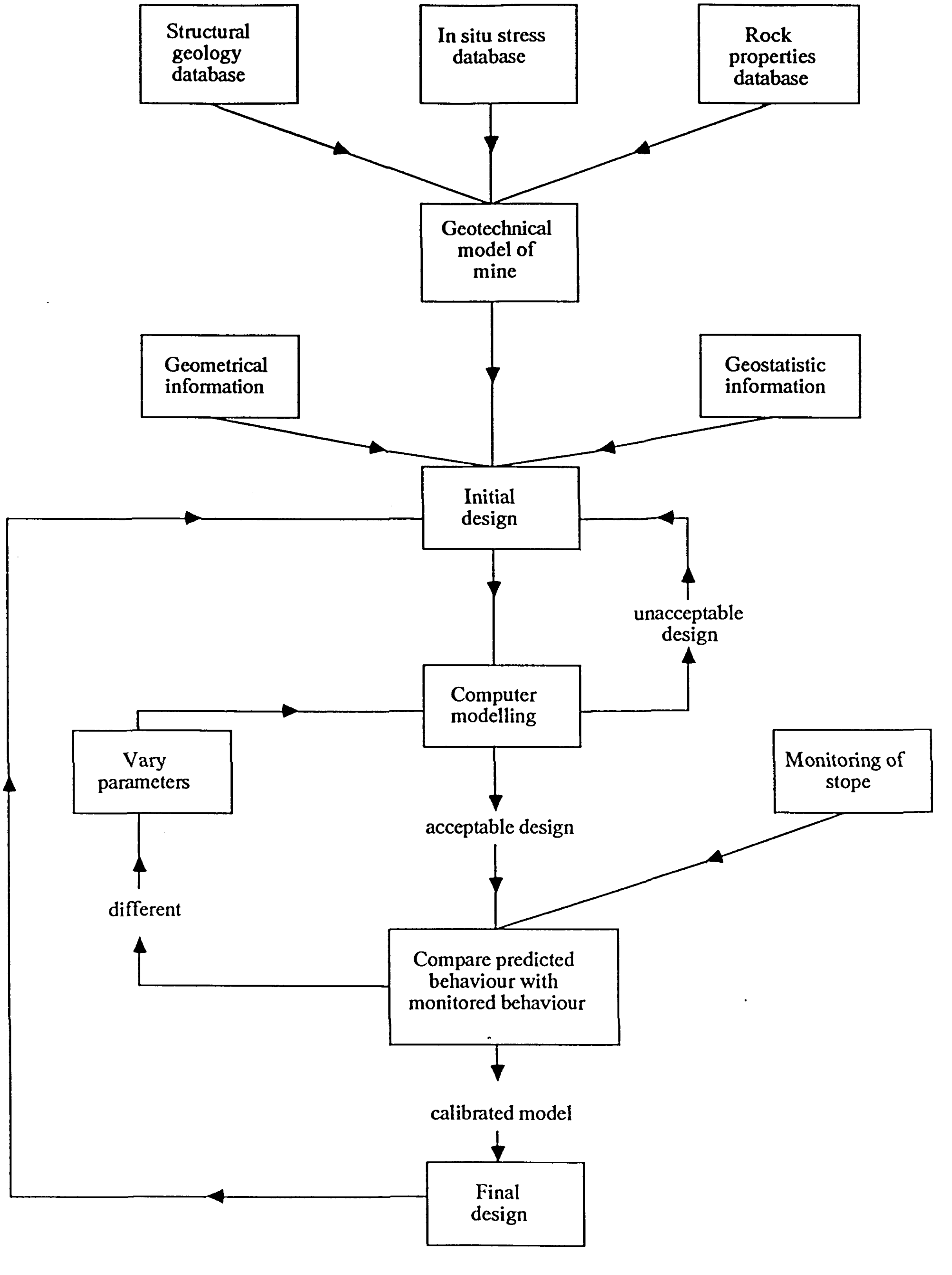


Figure 3.20 Advanced stope design methodology.

Measurement of *in situ* stress is well documented and many instruments have been developed for its measurement (Dyke, 1989). However, each measurement is only a point measurement, and the true stress distribution is heterogeneous. Thus it must be realised that a single measurement at a point may bear little relation to a volume average of the *in situ* stresses. Thus the aim is to derive a representative volume average of the *in situ* stress field by a series of *in situ* stress measurements throughout the rock mass. Care must be taken when averaging the stresses, as they are tensor values, not scalars, and the method described in Hyett *et al.* (1986) for averaging stress tensors is recommended.

The stress field measured is in general only pertinent to a particular lithological unit. The presence of large discontinuities, such as faults, has a marked effect on the stress distribution. Therefore, before beginning a course of *in situ* stress measurements, a geological plan of the area should be used to divide the rock mass into separate units, where the stress field can be reasonably assumed to be relatively homogeneous.

Large numbers of *in situ* stress measurements are both costly and time consuming, and few mines have carried out a detailed determination of their *in situ* stress field. Where no *in situ* stress results exist, great care must be taken when interpolating or estimating the stress field. Hoek and Brown (1980) presented a collection of *in situ* stress measurements throughout the world, showing the variation of vertical stress with depth, and the variation of the ratio of average horizontal to vertical stress with depth. The results showed that the vertical stress was commonly not a principal stress and not equal to the overburden weight, due to heterogeneity of the stress field. They also showed that there were wide variations in the ratio of horizontal stress to vertical stress. This arises from the complex load path and geologic history to which the rock mass was subjected. Use of the charts in Hoek and Brown (1980) is not recommended.

Where no *in situ* stresses are available it is preferable to perform design stress analysis using a range of *in situ* stresses. In such situations Brown (1985) suggested that the principal stresses should be assumed to be vertical and horizontal, that the vertical stress was due to the weight of the overburden, and that the major horizontal stress exceeded the vertical stress at depth. By a process of trial and error and the performance of existing or new excavations, the *in situ* stress field could be estimated.

However, overall, it is desirable to carry out a small series of *in situ* stress measurements, away from any mining operations, in order to try to determine the orientations, and possibly the ratio of the magnitudes, of the stresses. Then, as part of any analysis, the magnitudes of the stresses can be varied until the actual performance of the rock mass matches the predicted performance.

Rock Mass Properties

For the boundary element programs used in the design, the uniaxial compressive strength, Young's modulus and Poisson's ratio of the rock mass need to be known, for the prediction of elastic displacements. These displacements, in general, will bear little resemblance to overall displacement measurements, since most displacement is caused by movement on discontinuities, which is non-elastic. The intact Young's modulus and Poisson's ratio can be determined by performing uniaxial compression tests in the laboratory on samples of intact rock. These values will not represent the *in situ* values, which will be significantly lower than those measured in the laboratory.

Hoek (1988) proposed the use of an empirical equation linking Young's modulus, E, with RMR':

$$E = 10^{\left(\frac{RMR'-10}{40}\right)} \tag{3.5}$$

Equation (3.5) has been found to provide a reliable basis for estimating the modulus of deformation, E, of rock masses, particularly during the early stages of a project when relatively little field information is available.

The rating RMR' is not the same as the rock mass classification rating RMR due to Bieniawski (1976), as described in Section 3.4.1. RMR' ignores the joint orientation factor, as these values were determined from case studies on tunnels and do not apply to open stopes. The influence of joint orientation should be taken into account by any analysis. RMR' also sets the value of the groundwater factor to 10, equivalent to completely dry conditions, as the influence of groundwater pressure should be taken into account by any analysis.

The *in situ* strength of the rock mass is best defined using an empirical equation. The empirical equations given in Murrell (1965) and Hoek and Brown (1980) both take into account the influence of the triaxial state of stress in a rock mass.

Hoek and Brown (1980) developed the following equation:

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^2} \tag{3.6}$$

σ_1 = major principal stress at failure

σ_3 = minor principal stress

σ_c = uniaxial compressive strength of rock mass

m and s are constants which depend on the properties of the rock mass and the extent to which it has been broken. The uniaxial compressive strength of the rock mass is derived from the uniaxial compressive strength of intact rock using the equation:

$$\sigma_c(rock\ mass) = \sqrt{s} \cdot \sigma_c(intact\ rock) \tag{3.6b}$$

Hoek and Brown (1980) produced a chart showing a plot of m/m_i and s against the rock mass classification rating, RMR, due to Bieniawski (1976). m_i is the intact value for m, *i.e.* the value of m when $s = 1$, and varies according to rock type. This chart was widely accepted at the time and experience has shown that the estimated rock mass strengths were reasonable when used for slope stability studies in which the rock mass is usually disturbed and loosened by relaxation due to excavation of the slope. However, the estimated rock mass strengths generally appeared to be too low for application involving underground excavations where the confining stresses do not permit the same degree of loosening as would occur in a slope (Hoek, 1988). In order to account for this Hoek (1988) presented a new set of equations relating RMR' to m/m_i and s (RMR' being defined as above).

Disturbed Rock Masses :

$$\frac{m}{m_i} = \exp\left(\frac{RMR' - 100}{14}\right) \tag{3.7}$$

$$s = \exp\left(\frac{RMR' - 100}{6}\right) \tag{3.8}$$

Undisturbed or Interlocking Rock Masses :

$$\frac{m}{m_i} = \exp\left(\frac{RMR' - 100}{28}\right) \tag{3.9}$$

$$s = \exp\left(\frac{RMR' - 100}{9}\right) \tag{3.10}$$

Equations (3.7) to (3.10) have been used to construct Table 3.4 and Figure 3.21 shows a plot of these revised equations.

The values of m and s obtained from Table 3.4 or using equations (3.7) to (3.10) must be treated with care. For open stope design it is recommended that the undisturbed equations (Equations 3.9 and 3.10) be used for determining the constants for the Hoek-Brown failure criterion. However, this empirical strength criterion should only be used as a first estimate for rock strength, and the *in situ* rock strength can be found by back analysis of trial stoping operations (Brady, 1977) or by iterating as part of the observational cycle (Pariseau *et al.*, 1985).

Initial Layout

The layout of stopes in an orebody is highly subjective, with many factors to be taken into account. The layout of stopes in the lead/zinc orebodies at Mount Isa mine, Australia provide a good example of how stopes should be laid out in closely spaced orebodies (Figure 3.22). The stope and pillar layout is an iterative process. Once the most suitable sites for the stopes and pillars have been identified then a preliminary design for the stope layout can be established. At this stage the layout needs to be checked to see whether it meets scheduling and engineering constraints such as size of stope, stope width, stope height, distance to nearest ore pass, *etc.* The next stage of the design involves the use of the simple elastic boundary element programs, that were described in Section 2.3.5.

Design Analysis

The type of boundary element analysis required is dictated to a large degree by the design problem. If a global assessment is required of a proposed stope layout, then a displacement discontinuity program will probably be the preferred program, especially if the orebodies are narrow. This is because the displacement discontinuity method only requires the hangingwall of the stope to be discretised.

Disturbed rock mass m and s values *undisturbed rock mass m and s values*

EMPIRICAL FAILURE CRITERION $\sigma_1 = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^2}$ σ_1 = major principal stress σ_3 = minor principal stress σ_c = uniaxial compressive strength of intact rock, and m and s are empirical constants.		CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ANENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS & METAMORPHIC CRYSTAL-LINE ROCKS - *amphibolite, gabbro gneiss, granite, norite, quartz-diorite*
INTACT ROCK SAMPLES						
Laboratory size specimens free	m	7.00	10.00	15.00	17.00	25.00
from discontinuities	s	1.00	1.00	1.00	1.00	1.00
CSIR rating: RMR = 100	*m*	*7.00*	*10.00*	*15.00*	*17.00*	*25.00*
NGI rating: Q = 500	*s*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
VERY GOOD QUALITY ROCK MASS						
Tightly interlocking undisturbed rock	m	2.40	3.43	5.14	5.82	8.56
with unweathered joints at 1 to 3m.	s	0.082	0.082	0.082	0.082	0.082
CSIR rating: RMR = 85	*m*	*4.10*	*5.85*	*8.78*	*9.95*	*14.63*
NGI rating: Q = 100	*s*	*0.189*	*0.189*	*0.189*	*0.189*	*0.189*
GOOD QUALITY ROCK MASS						
Fresh to slightly weathered rock, slightly	m	0.575	0.821	1.231	1.395	2.052
disturbed with joints at 1 to 3m.	s	0.00293	0.00293	0.00293	0.00293	0.00293
CSIR rating: RMR = 65	*m*	*2.006*	*2.865*	*4.298*	*4.871*	*7.163*
NGI rating: Q = 10	*s*	*0.0205*	*0.0205*	*0.0205*	*0.0205*	*0.0205*
FAIR QUALITY ROCK MASS						
Several sets of moderately weathered	m	0.128	0.183	0.275	0.311	0.458
joints spaced at 0.3 to 1m.	s	0.00009	0.00009	0.00009	0.00009	0.00009
CSIR rating: RMR = 44	*m*	*0.947*	*1.353*	*2.030*	*2.301*	*3.383*
NGI rating: Q = 1	*s*	*0.00198*	*0.00198*	*0.00198*	*0.00198*	*0.00198*
POOR QUALITY ROCK MASS						
Numerous weathered joints at 30-500mm,	m	0.029	0.041	0.061	0.069	0.102
some gouge. Clean compacted waste rock	s	0.000003	0.000003	0.000003	0.000003	0.000003
CSIR rating: RMR = 23	*m*	*0.447*	*0.639*	*0.959*	*1.087*	*1.598*
NGI rating: Q = 0.1	*s*	*0.00019*	*0.00019*	*0.00019*	*0.00019*	*0.00019*
VERY POOR QUALITY ROCK MASS						
Numerous heavily weathered joints spaced	m	0.007	0.010	0.015	0.017	0.025
<50mm with gouge. Waste rock with fines.	s	0.0000001	0.0000001	0.0000001	0.0000001	0.0000001
CSIR rating: RMR = 3	*m*	*0.219*	*0.313*	*0.469*	*0.532*	*0.782*
NGI rating: Q = 0.01	*s*	*0.00002*	*0.00002*	*0.00002*	*0.00002*	*0.00002*

Table 3.4 Revised approximation of the relations between rock mass quality and material constants for the Hoek–Brown failure criterion (after Hoek, 1988).

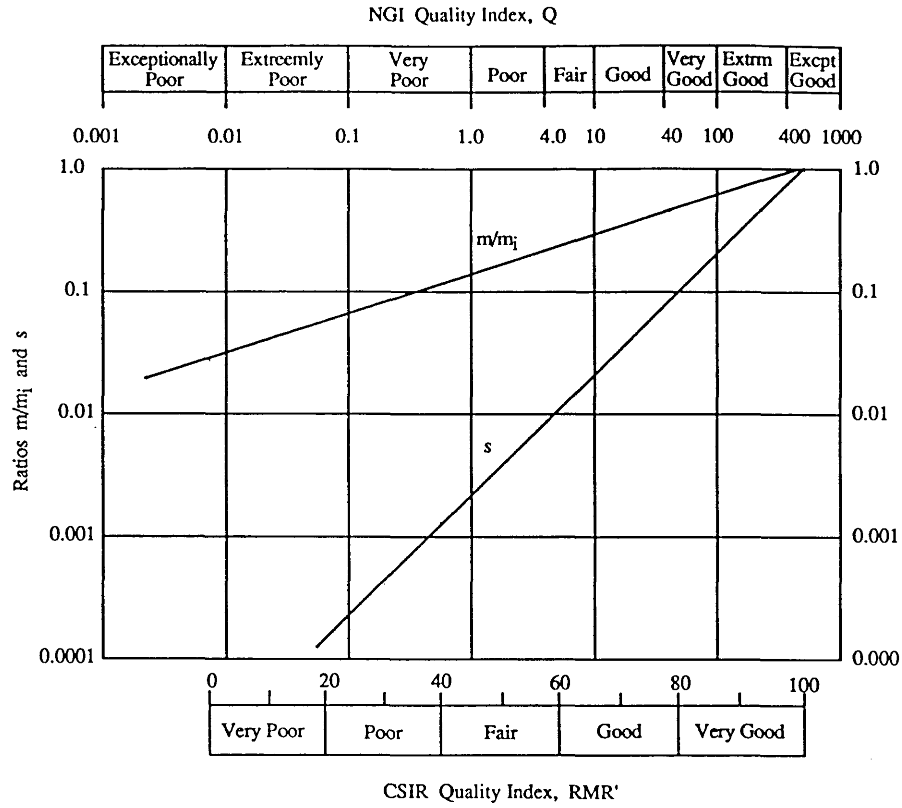


Figure 3.21 Plot of the ratio m/m_i and s against rock mass quality using the revised equations of Hoek (1988).

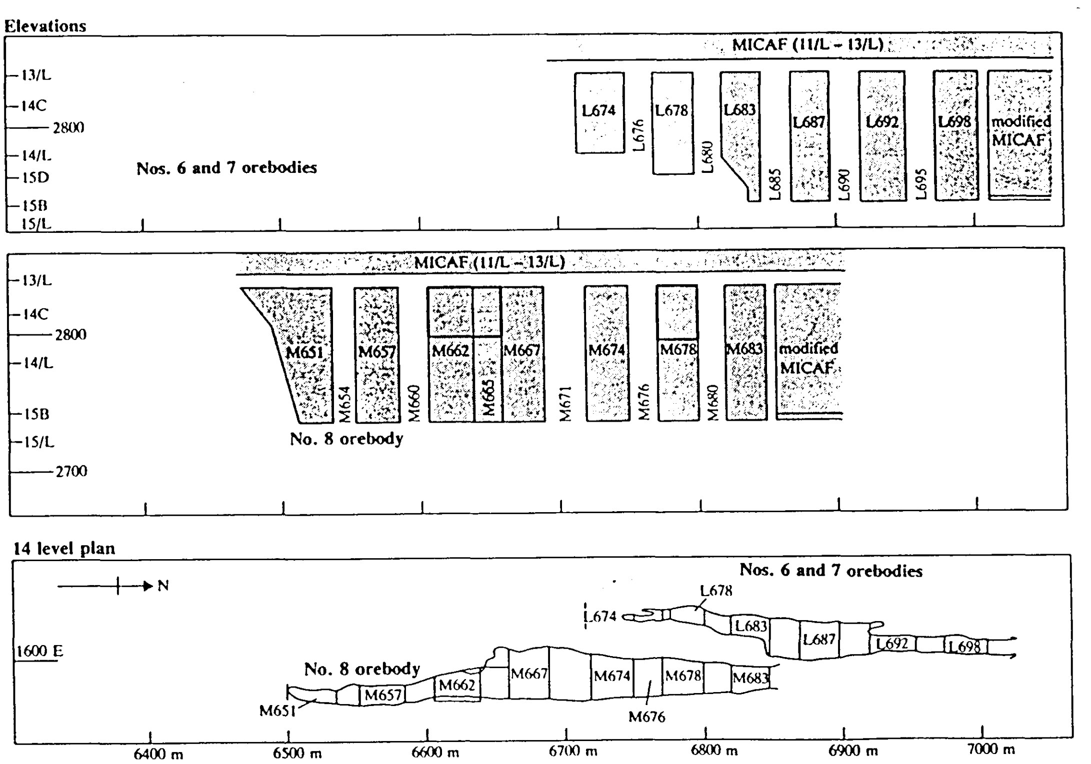


Figure 3.22 Stope layout in the 6/7 and 8 orebodies at 13–15 level, Mount Isa mine, Australia (after Brady and Brown, 1985).

This allows a larger number of stopes to be modelled compared to other types of boundary element program, which is important with regard to the limited memory capacity of micro-computers. The displacement discontinuity method is also useful for the modelling of closely spaced parallel orebodies, which lie within each others zones of influence. If two-dimensional plane strain conditions apply, programs such as TWODD, and where more modelling facilities are required, MINAP, are perfectly adequate. If a complete strain analysis is required the program TAB4 is recommended.

For more massive orebodies, simple indirect boundary element programs can be used, although the need to model the hangingwall, footwall, roof and floor of all stopes leads to a reduction in the number of stopes that can be modelled at once using a micro-computer. For two-dimensional plane strain analysis the programs BOUND and TWOFS are adequate, but if complete plane strain analysis is required then BEM11 must be used. The simplest type of analyses to undertake are two-dimensional plane strain analyses of vertical or horizontal sections of an orebody. If the stope layout cannot be considered to be amenable to plane strain analysis then three dimensional analyses must be used. The program MINTAB is generally acceptable for this type of analysis. Further details on the individual merits of these programs, together with advice on their use, is given in Section 2.3.5.

If the design problem is to model the behaviour of individual stopes and pillars then displacement discontinuity programs are of little use, as they do not allow the stress distribution within pillars to be determined. This is because the method implicitly assumes the orebody is of zero thickness, and hence the stress distribution is constant across a pillar, which is not the case in reality. In these situations, where plane strain conditions exist, simple or complete plane strain boundary element programs are suited. If the orebody is narrow and has close parallel boundaries, then care must be taken when discretising the boundary. It is important to use sufficient elements to model the rapid changes in stress along the hangingwall, and the element length should not be more than the orebody thickness to prevent problems with convergence in the numerical analysis.

Once the stress distribution in the rock mass has been modelled, then it is possible to plot areas of tensile stress, and areas where the compressive stresses exceed the rock mass strength. The design is then refined to eliminate unacceptable states of stress, taking into account the required service role and life of the stope, or until a compromise is reached between geotechnical and other engineering constraints. It may be necessary to accept unfavourable states of stress in some areas in order to produce a layout that is economic.

For the case of narrow, tabular orebodies, the stress analysis is likely to have been conducted on a vertical cross-section of the orebody. This is because tabular orebodies are usually worked at regular intervals and mined along strike, and are commonly amenable to two-dimensional plane strain analysis. Figure 3.23 shows a typical layout of longitudinal longhole mining in a tabular orebody. Dimensioning the strike span of such stopes is important, as dilution increases with increasing stope span. There is a critical strike length for all stopes beyond which increased dilution makes stoping uneconomic. Three-dimensional stress analysis using programs like MINTAB are unable to predict the stability of the hangingwall and hence dilution levels, and can only give an indication of the extent of the tensile zone developed in the hangingwall.

The stability graph method is the most effective way to dimension these stope hangingwalls. The hangingwall is evaluated using the methodology outlined in Section 3.4.1, which incorporates the effect of the stress distribution in the hangingwall, to determine a stability number, N. From the stability graph (Figure 3.11) the corresponding maximum safe shape factor, S, can then be determined. Then, for a given mining level interval, the maximum safe strike stope span can be calculated as shown in Figure 3.24. This maximum span may change as mining progresses and more information becomes available, allowing the updating of the input parameters to the stability graph method or to the stress analysis. The design engineer then has to decide the level of dilution that is acceptable for the stope, or the allowable factor of safety against failure, in order to determine what the actual stope span should be.

The stope and pillar mining sequence must be geomechanically sound but must also satisfy scheduling requirements with regard to ore output, material handling, and manpower and equipment availability. The general principle used for stope sequencing is to try to mine the higher grade blocks of ground as soon as possible, in order to offset the high development costs associated with open stope mining. This does not mean that the sequence is dictated by grade alone. The mining block is considered as a whole and stope blocks that have little intrinsic

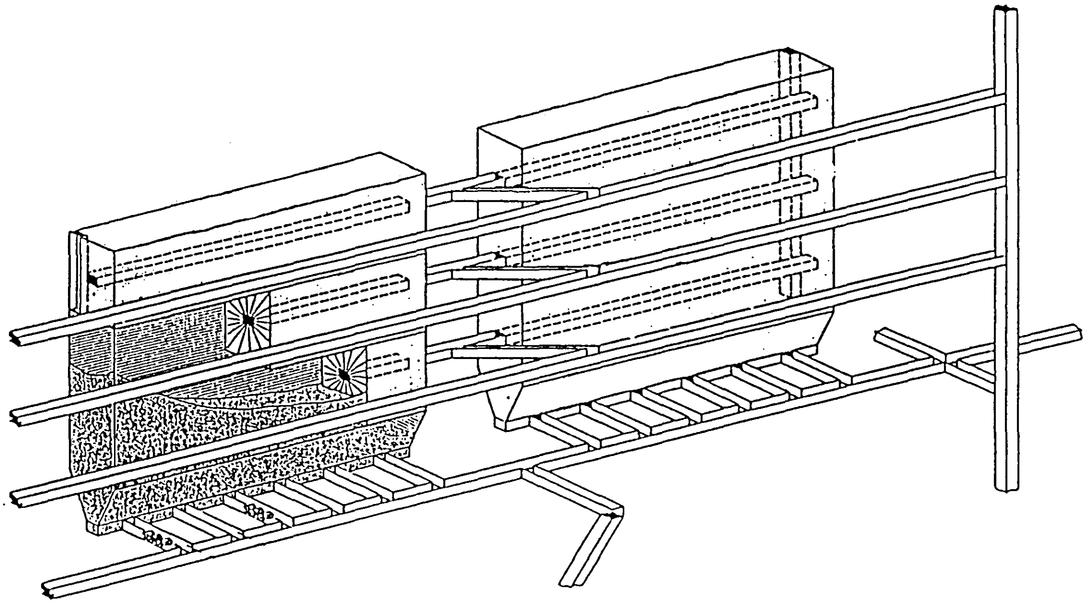
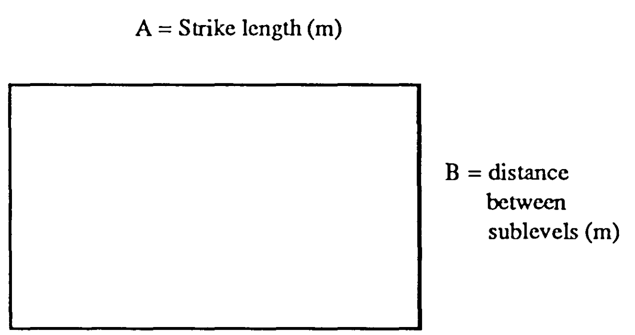


Figure 3.23 Typical longitudinal longhole stoping layout in a tabular orebody.



Assume stability number, N, is known.

From the Stability Graph (Figure 3.11) the maximum allowable value for the shape factor, S, can be determined.

The shape factor, S, is defined as the area / perimeter.

Hence $S = \dfrac{A \times B}{2\,(A \times B)}$

Set either A or B and calculate the other dimension using the equation above.

Figure 3.24 Calculation of maximum stope dimensions using the shape factor, S.

integrity or support potential should be mined first. In this way mining progresses from less stable rock to more stable rock, such that as the stress levels increase, due to the removal of the support provided by the ore, the stress is concentrated in rock that is more able to sustain the stress. The extraction sequence should also try to avoid the generation of narrow remnants, such as those developed when a pillar is gradually reduced in size. Such pillars should be mass-blasted as the high stress levels that build up in such remnants lead to dangerous ground conditions.

In order to determine the rock mass response around stopes due to the proposed extraction sequence, stress analysis programs need to be able to model the sequence accurately. This can be accomplished using a two-dimensional displacement discontinuity program such as MINAP, or a three-dimensional program such as MINTAB. These programs have the ability to model progressive mining of an orebody, including the placement of backfill as part of the mining cycle. It is also essential to remember that major installations and excavations that are away from the orebody need to be protected. This includes major haulageways, ventilation shafts, ore passes and main shafts, which, although they are all affected to some degree by mining, must be kept serviceable for as long as they are required.

Rock Performance Monitoring

Once the proposed mining block has been initially modelled, trial stoping is begun and the stopes are monitored to detect stress and displacement changes. Monitoring is essential to gain an understanding of the bulk strength (including reinforcement) of the rock mass surrounding stopes, and in crown and rib pillars, as well as the nature of the loadings due to mining. Monitoring of stress changes is usually achieved using CSIRO hollow inclusion cells or vibrating wire stressmeters, and displacement measurements are usually from rod or wire extensometers. Microseismic monitoring has been shown to have potential for given warnings of imminent failure and possibly as a means of giving long range forecasts. Worotnicki *et al.* (1980) concluded that microseismic techniques were probably the most cost effective way of monitoring extensive areas. Further discussions on monitoring programs undertaken at open stoping operations are given in Brady and Brown (1985); Lee and Bridges (1981); Krauland (1985); Alexander *et al.* (1981).

Retrospective Analysis

Using the information from instrumented monitoring, together with visual observation of failure mechanisms and rock performance, retrospective analysis of the open stope layout can be undertaken. The object is to match observed behaviour with that predicted by the computer analysis to produce a calibrated model (Pariseau *et al.*, 1985). If monitoring has revealed inadequacies in the original conceptualisation of the rock mass, then a new conceptual model has to be devised and the appropriate analysis selected. In many cases the analysis of local failures can reveal inaccuracies in the input *in situ* stress field, or give information on the *in situ* strength parameters (Brady, 1977). This retrospective analysis can be seen as the feedback loop in Figures 3.16 and 3.20.

Once the computer analysis can be shown to be modelling the rock mass behaviour in an acceptable manner, and that back analysis has allowed the model to be calibrated to site conditions, then the model can be used as a predictive tool. During the mining of the production stopes monitoring is maintained, to detect unforeseen instabilities around the stopes, and to check whether the model continues to represent reality. If monitoring detects any major instabilities that had not been predicted, or were not modelled, then complete re-analysis may be necessary using a different numerical program, or recalibration of the model may suffice. The observational nature of this design methodology means that the characteristics of the rock mass – the *in situ* stress field and the behaviour of the rock mass around open stopes – are constantly being re-evaluated.

CHAPTER 4
OPEN STOPE PILLAR DESIGN

4.1 Introduction

Open stope mining, as one of the naturally supported mining methods (Figure 3.1), seeks to control rock mass displacements throughout the zone of influence of mining, whilst mining proceeds (Brady and Brown, 1985). This requires local stability around stopes, as well as the general control of displacements in the-near field rock. These can be considered as two separate design issues (Figure 4.1), although there is a great degree of interaction between them. Chapter 3 discussed open stope design, and developed a methodology for the design of stable stope dimensions, stope layouts and stope sequencing. This chapter is concerned with near-field ground control, which is achieved by the provision of support, usually in the form of pillars, between excavations. Pillars also divide stope blocks into individual stopes, giving flexibility in ore extraction by producing multiple working faces.

The removal of the support effect of orebody rock by mining, results in an increase in the elastic strain energy stored in the near-field rock. The objective of pillar design is to ensure that the sudden release of this strain energy cannot occur. If this energy was suddenly released, then pillars could rupture, stopes could close, and there could be a rapid generation of fractures in the near-field rock. Such events would damage the immediate working area, as well as adjacent areas of the mine, and are a hazard to personnel. The effective performance of pillars is related to their dimensions and their layout within an orebody. The important factors to be taken into account are the bulk strength of pillars (including reinforcement) and the stress distribution within them.

Pillars that are located in the orebody result in either temporary or permanent sterilisation of ore reserves, with a resulting loss in revenue. Unnecessarily large pillars also result in a loss of ore reserves, or the need to recover pillars subsequently

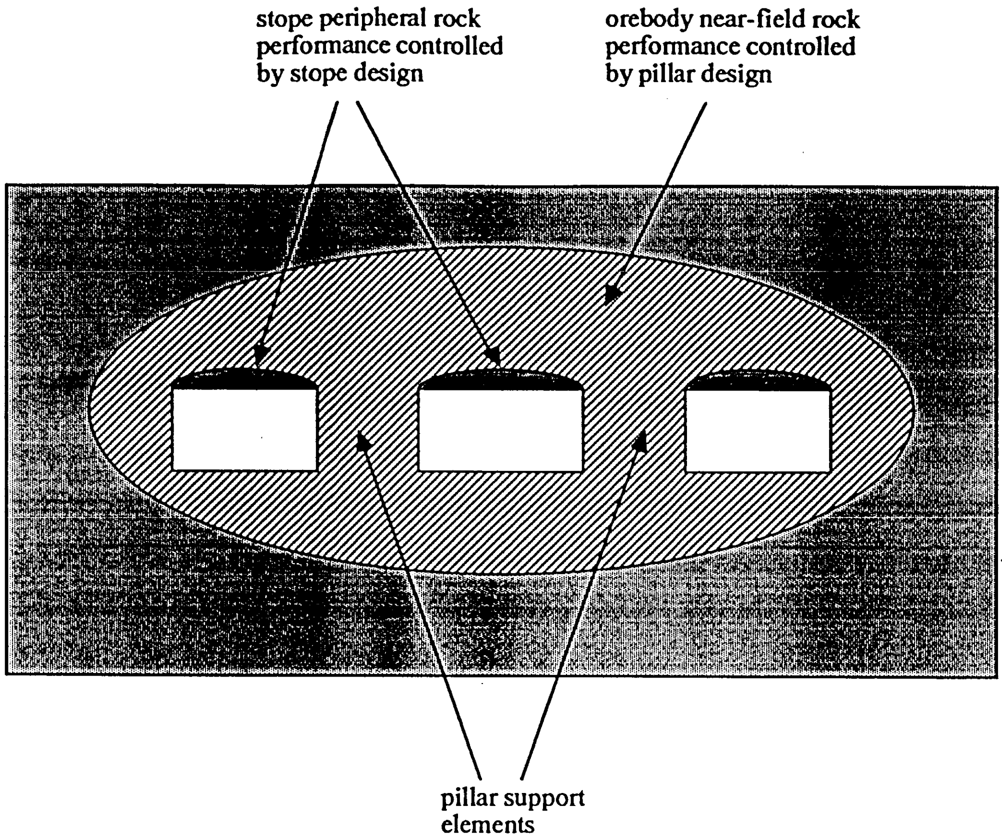


Figure 4.1 Schematic illustration of problems of mine near-field stability and stope local stability, affected by different aspects of mine design (after Brady and Brown, 1985).

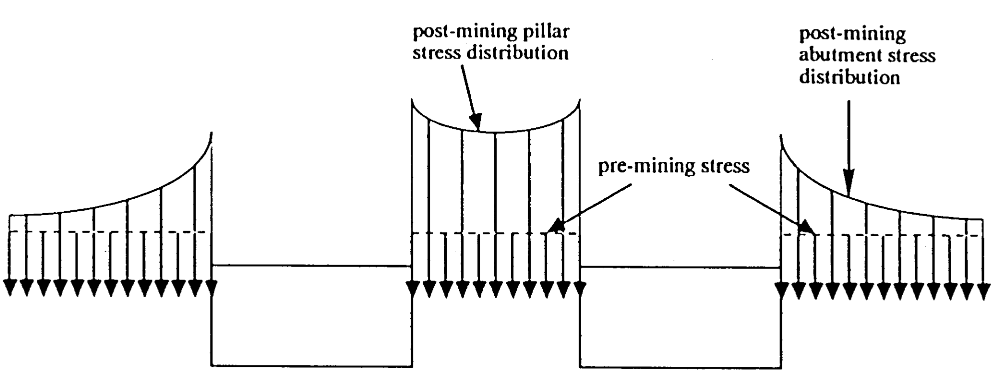


Figure 4.2 Redistribution of stress in the axial direction of a pillar accompanying stope development (after Brady and Brown, 1985).

by a more costly mining method. If pillars are to be excavated at a later date then care must be taken to ensure that they are in a condition conducive to efficient extraction. They should be large enough, so as not to be too highly stressed, or they should be designed to fail partially, leading to a relaxing of stresses, whilst maintaining the integrity of the pillar.

The need to optimise the dimensions of crown pillars, whilst ensuring their stability, has become a major concern for the mining industry (Brown, 1985). This is especially true with the increasing use of open stoping for the mining of metalliferous orebodies (Bridges, 1983). By the very nature of the mining method, once pillar dimensions have been selected it is relatively difficult and expensive to change the design. This is in contrast to overhand cut-and-fill mining, where there is often scope for deciding on pillar thickness during the mining operation, based on progressive assessment of the stability of the material remaining above the stope back.

Crown pillar dimensioning in open stopes has to date, relied largely on experience of past pillar performance. Some empirical techniques for pillar design have been developed, which are discussed in Section 4.3, as well as more modern numerical and analytical techniques. A rational rock mechanics approach to crown pillar design needs to establish an estimate of the bulk strength (including reinforcement) of pillars, as well as the nature of loadings generated as a result of mining. In Section 4.4, an advanced design methodology for crown pillars in narrow, steeply dipping, tabular orebodies is developed.

4.2 Pillar Failure Mechanisms

Pillars can be designed to be temporary or permanent, yielding or non-yielding. The failure of a pillar design, which leads to unstable conditions and possibly uncontrolled pillar collapse, is also dependent on the role of the pillar. In many cases the collapse of a pillar is quite acceptable, but the collapse of a regional support pillar or waste barrier pillar could be disastrous. To predict pillar stability, a design must identify the mechanisms involved in pillar failure. Pillar failure can be divided into two basic modes:

- stable, progressive failure
- unstable, bursting failure.

Stable failure is characterised by gradual deterioration of a rock mass in a relatively slow and non-violent manner. Unstable failure is associated with the violent release of energy in sudden bursts, causing instantaneous failure of a rock mass.

In mine design, the aim is to design pillars which, if they fail, fail in a stable, progressive manner. Failure here is equated to a loss of load bearing capacity, which in turn is associated with a loss of pillar confinement. Rock at the extremities of a pillar provides the central core with confinement. One of the first visual signs of pillar deterioration is the development of fractures at the pillar boundaries, which results in the loss of some of the original confinement. With increased loads, the pillar begins to expand laterally, slabbing occurs, and fracturing propagates to the core of the pillar. The fracturing has to be quite severe before blocks, formed by the fracturing, fall out and pillar disintegration begins.

Visual observation of pillars in room and pillar mines is common (Krauland and Soder, 1987). Detailed observations and measurements have been made by many workers (Bunting, 1911; Greenwald *et al.*, 1941; Wagner, 1974; van Heerden, 1975; Hardy and Agapito, 1975) and they have confirmed the pillar failure mechanisms described above. Observation of crown pillar failure in open stopes is not common, due to lack of access for visual observation, but Brady (1977) reported that the response of open stope pillars resembled that of pillars in room and pillar mines. This has been confirmed subsequently by monitoring of crown pillars (Worotnicki *et al.*, 1980). Table 4.1 lists the indirect signs that indicate that a pillar has stability problems. One of these signs on its own does not necessarily indicate pillar failure, but they are commonly reported during pillar failure.

The process of extracting ore from open stopes results in stress redistribution, and an increase in pillar loading (Figure 4.2). When the state of stress in a pillar is less than the *in situ* rock mass strength, the pillar remains intact and responds elastically to the increased state of stress. In mining, interest is usually centred on the peak bulk strength of a pillar, up to the point where pillar rupture first occurs. Subsequently, interest is related to the post-peak performance of the pillar.

1. Cracking and spalling of rock in pillar development and raises.
2. Audible noises heard in the pillar or located with microseismic systems.
3. Deformed or plugged drillholes.
4. Overdraw from stopes due to spalled pillar rock.
5. Stress redistribution from rib pillars affecting nearby pillars, and hangingwall and footwall drives and raises.
6. Hourglassing of pillars.
7. Major displacements and changes in stress measured by instrumented monitoring systems.

Table 4.1 Indirect signs indicating pillar stability problems

The behaviour of a pillar subject to mining induced load is determined by several factors:

- dimensions of the pillar
- structural geology of the pillar rock mass
- nature of the contact between pillar and country rock.

In relatively massive rock the dominant pillar failure mechanism is spalling (Figure 4.3a). This leads to hourglassing of the pillar and is one of the first visual signs of overstressing in a pillar. Pillars with large width to height ratios, such as crown pillars in narrow, tabular orebodies, invariably fail in this manner. Where the width to height ratio is lower, failure due to the formation of inclined shear fractures transecting the pillar are more common (Figure 4.3b). Where soft partings exist between the pillar and the country rock, then yielding of this soft material generates transverse stresses over the pillar end surfaces, which can lead to internal splitting of the pillar (Figure 4.3c).

In jointed rock masses, the structure of the rock mass exerts a dominant influence on the pillar failure mechanism. A pillar with a set of transgressive discontinuities (Figure 4.3d) can be expected to yield if the angle of inclination of the fractures to the pillar principal plane (that perpendicular to the pillar axis) exceeds their effective angle of friction. A pillar with a well developed foliation or schistosity parallel to the principal axis of loading will fail in a buckling mode (Figure 4.3e).

4.3 Previous Approaches to Pillar Design

4.3.1 Introduction

Pillar design in open stopes has been, in the past, by precedent practice – that is, pillar design was based on the experience and observation of the performance of other operations in similar geotechnical conditions to those around the orebody of interest.

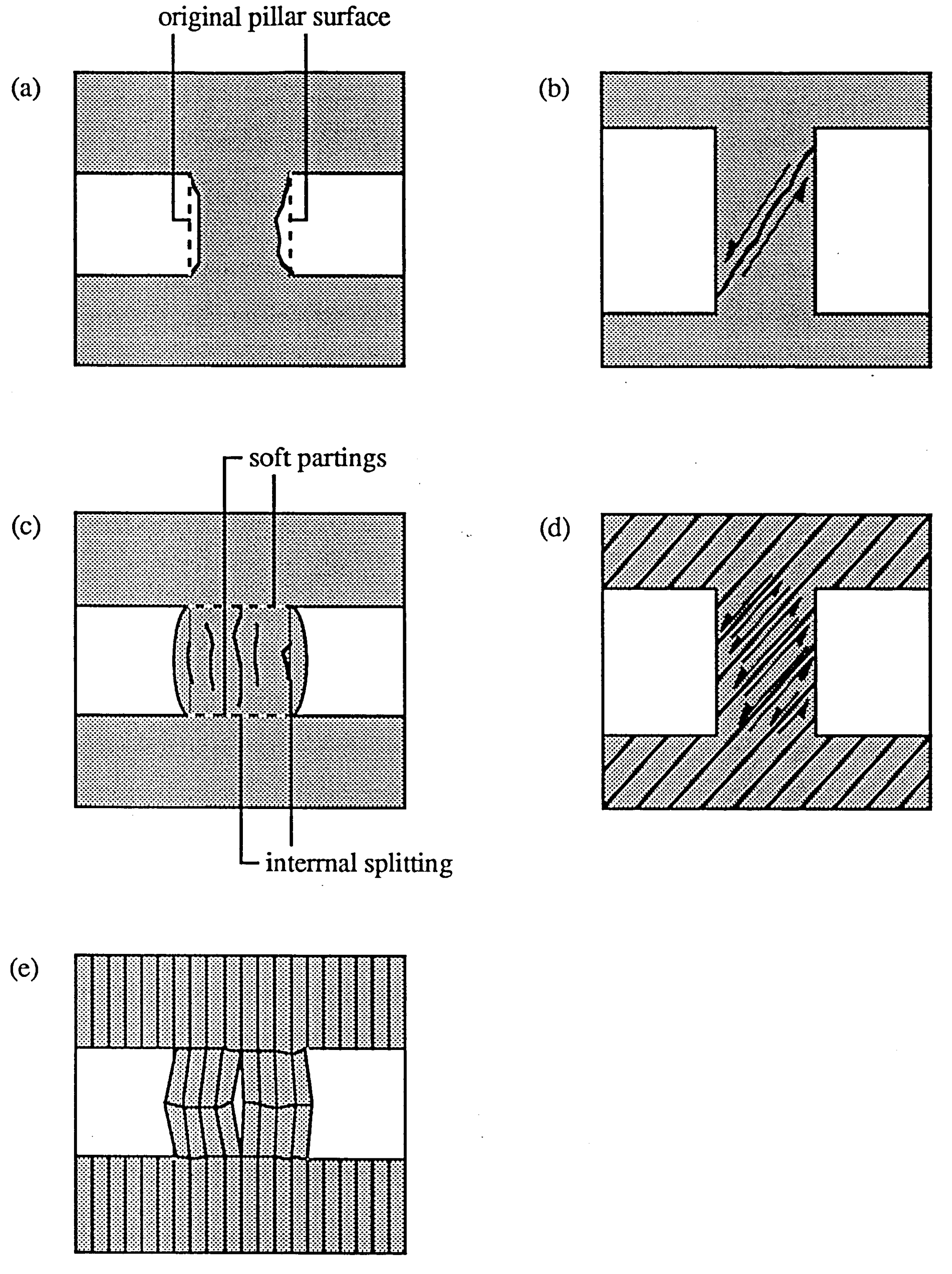


Figure 4.3 Principal modes of deformation behaviour of mine pillars (after Brady and Brown, 1985).

In an attempt to establish a more appropriate basis for the design of pillars in open stopes, three approaches have been identified from the literature:

- empirical design
- numerical analysis design
- global stability design.

Historically, empirical methods were the only method available for the design of mine pillars, and involved the comparison of pillar strength with pillar load. More recently, with the increase in availability of computers, numerical analysis design has become more popular. The third design technique entails the consideration of pillar stiffness, and has been rarely used in practice due to the lack of data on *in situ* rock stiffnesses.

The numerical design method is a precise method from a mathematical viewpoint, but problems arise in the selection of input parameters for the programs. The complex rock mass has to be simplified for modelling purposes, and it is essential that the failure mechanisms of pillars in the rock under consideration are properly accounted for in the analysis. These problems have resulted in practical mine pillar design still relying primarily on the first design method (Logie and Matheson, 1982).

4.3.2 Empirical Design

Logie and Matheson (1982) reviewed empirical pillar design equations and found that although the formulae differed slightly in form, they had two basic components in common: the size-strength effect and the shape-strength effect.

Size-Strength Effect

Rock, given a constant shape, decreases in strength as the size of the rock sample increases (Bieniawski and van Heerden, 1975), as Figure 4.4 demonstrates. The reason for this phenomenon is that as the rock sample is increased in size, there are an increasing number of discontinuities present in the sample. Thus the strength of a small laboratory 50 mm diameter specimen can be considered to be the intact rock strength, whereas the strength of a 2 m cube specimen would approximate to the *in situ* rock mass strength.

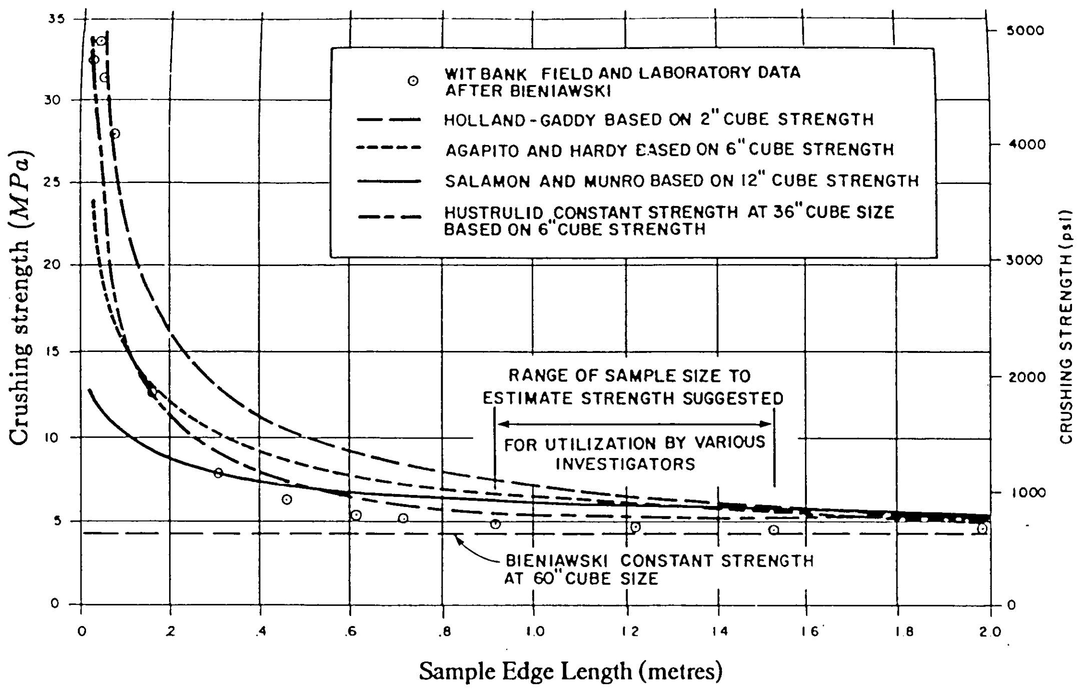


Figure 4.4 Comparison of various size–strength effect assumptions on data from Witbank coal (after Logie and Matheson, 1982).

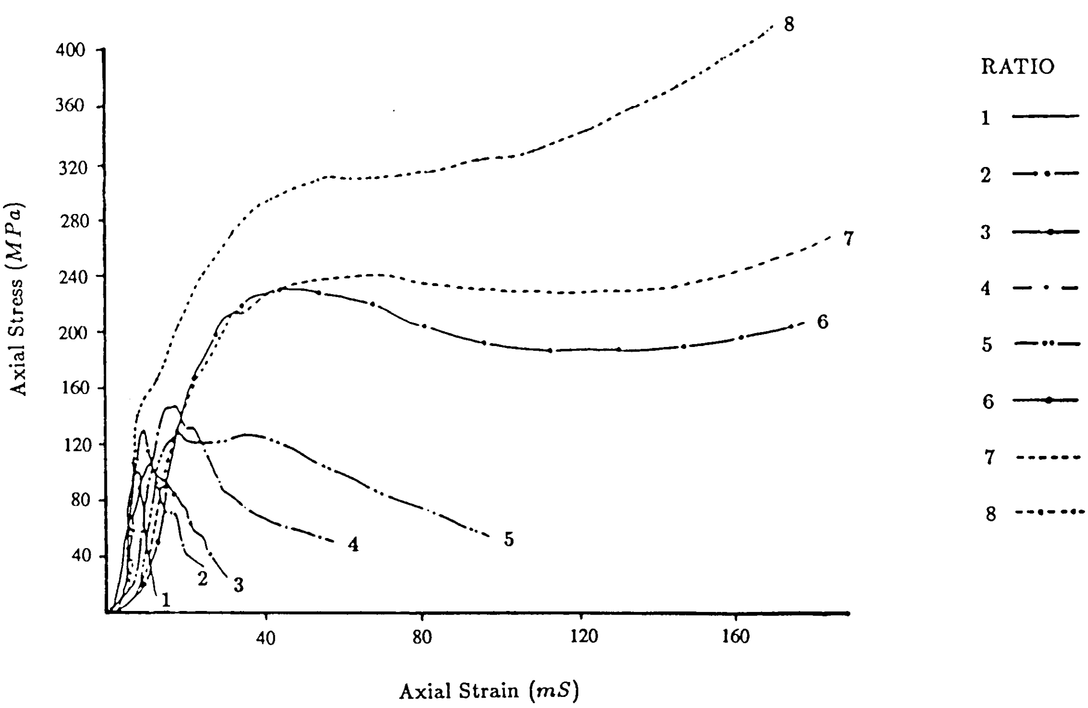


Figure 4.5 Stress–strain curves for cylindrical sandstone specimens with width to height ratios from 1 to 8. Specimen diameter 24 mm (after Wagner and Madden, 1984).

Shape-Strength Effect

The shape-strength effect has been reported by many workers (Bauschinger, 1876; Holland, 1964). The most widely known of the equations which describes this phenomenon is:

$$\frac{\sigma_p}{\sigma_c} = 0.778 + 0.222\left[\frac{W}{H}\right] \tag{4.1}$$

σ_p = compressive strength of a prism

σ_c = compressive strength of a prism for which $W = H$

W = minimum lateral dimension

H = prism height.

Equation (4.1) was originally developed by Bauschinger (1876). The shape-strength effect arises from three sources:

- confinement, which develops in the body of a pillar due to constraint on its lateral dilation

- redistribution of field stress components in the pillar

- change in pillar failure mode.

For testing the shape-strength effect of rock, the sample height is kept constant, whilst the sample width is increased. Wagner and Madden (1984) conducted a series of tests on sandstone specimens. Figure 4.5 shows that the strength of squat specimens increases rapidly with increasing width to height ratio. Wagner and Madden (1984) also showed that the failure mechanism changed with changes in width to height ratio. With slender samples, (*i.e.*, samples with low width to height ratios), the samples tended to fail violently in a brittle manner, whereas for squat samples, (*i.e.*, samples with high width to height ratios), failure is gradual and non-violent.

Pillar Strength Formulae

All subsequent work with empirical design methods has been concerned with the derivation of pillar strength formulae. Obert and Duvall (1967) produced a pillar formula that was a direct utilisation of the shape-strength effect, as developed by Bauschinger (1876). This equation does not include a size-strength relationship which leads to overestimation of pillar strength, and hence requires high factors of safety. A modified version which uses the uniaxial compressive strength for a minimum 2 m specimen has generally been found to be comparable to other design methods (Figure 4.6).

Salamon and Munro (1967) compiled a list of 98 stable and 27 collapsed pillars in South African coal mines. Using statistical analysis they developed the following coal mine pillar equation:

$$\sigma_p = \sigma_c \left(\frac{W^{0.46}}{H^{0.66}} \right) \tag{4.2}$$

σ_p = pillar compressive strength in psi

σ_c = compressive strength of a 1 $ft.$ cube of coal in psi

W = pillar least width in inches

H = pillar height in inches.

Where strength data were not available for a 1 $ft.$ cubic sample of coal, they suggested the following formula to scale up laboratory results:

$$\sigma_p = \sigma_s \left(\frac{W_p}{W_s} \right)^{0.46} \left(\frac{H_p}{H_s} \right)^{-0.66} \tag{4.3}$$

σ_p = pillar compressive strength in psi

σ_s = compressive strength of a cubic sample of coal of width W_s and height H_s in inches

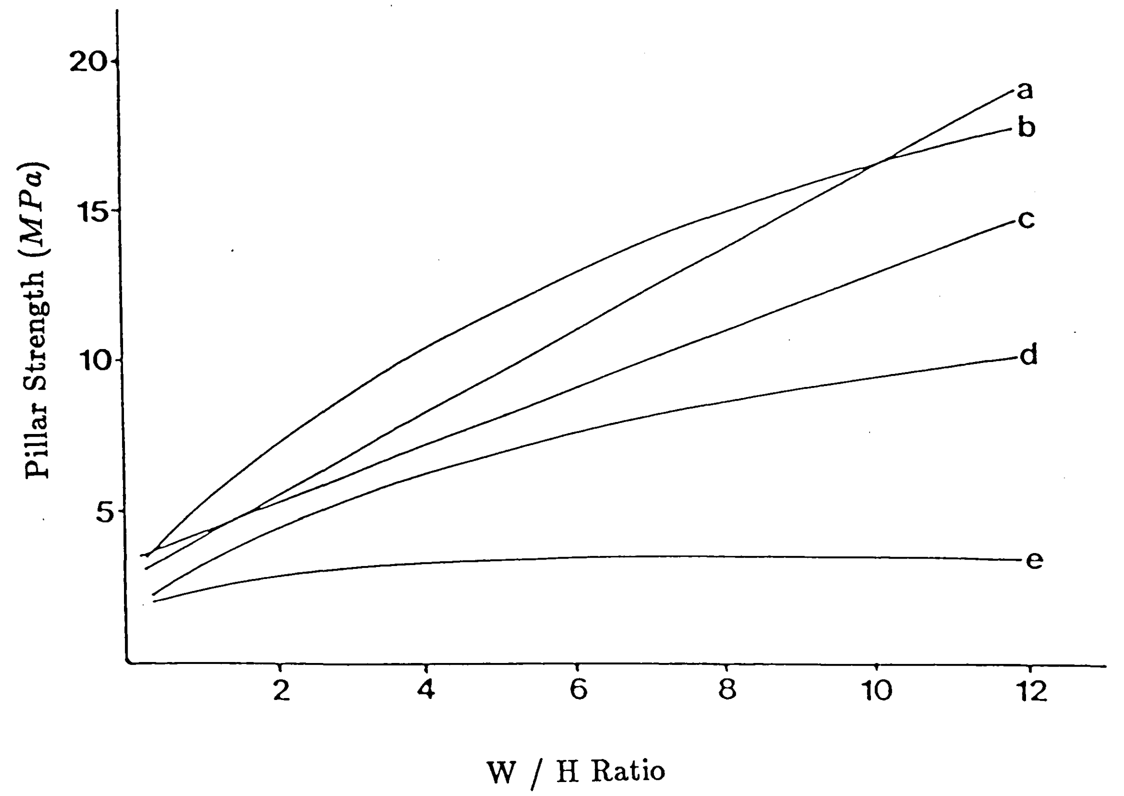
W_p = pillar width in inches

H_p = pillar height in inches.

Equations (4.2) and (4.3) are based on past experience in South African coal mines and should not be related to other deposits, except where the deposit is assumed to be similar geologically.

Hardy and Agapito (1975) suggested a modification to equation (4.3) for use in an oil shale mine based on a statistical analysis of observed laboratory and field strengths of coal and oil shale.

$$\sigma_p = \sigma_s \left(\frac{W_p}{W_s} \right)^{0.597} \left(\frac{H_p}{H_s} \right)^{-0.951} \tag{4.4}$$



(a) Bieniawski (σ_c of 60 inch cube = 4.3 MPa)

(b) Holland and Gaddy (σ_c of 2 inch cube = 31.2 MPa)

(c) Modified Obert and Duvall (σ_c of 60 inch cube = 4.3 MPa)

(d) Salamon and Munro (σ_c of 12 inch cube = 7.8 MPa)

(e) Wilson (σ_c of 60 inch cube = 4.3 MPa, assumed depth of coal = 120 $ft.$)

Figure 4.6 Comparison of the most widely used pillar formula. Data taken for a 6 $ft.$ seam of coal at Witbank colliery, South Africa (after Logie and Matheson, 1982).

Equations (4.1) to (4.4) are limited to relatively small ranges of width to height ratios, and must be adjusted for very large pillars, as the size-strength effect is not taken into account. These equations also indicate a decreasing rate of strength increase for greater width to height ratios, and do not take into account the strengthening effect of increased confinement with increased width to height ratios. Agapito and Hardy (1982) suggested a scaling relation, to relate specimen strength to pillar strength, such that the difference between the pillar volume, V_p, and the specimen volume, V_s, was taken into account.

$$\sigma_p = \sigma_s \left(\frac{V_p}{V_s}\right)^{-0.118} \left[\left(\frac{W_p}{H_p}\right) \Big/ \left(\frac{W_s}{H_s}\right)\right]^{0.833} \tag{4.5}$$

where subscripts p and s refer to the pillar and specimen, respectively.

These formulae have been developed for square pillars and have to be modified for rectangular pillars, such as crown pillars. Wagner (1974) reported that large rectangular specimens have the same strength as square specimens with side length equal to an *effective width*, which is given by:

$$Effective\ Width = 4A\,/C \tag{4.6}$$

A = pillar area
C = pillar circumference.

Kersten (1984) used Wagner's (1974) modified version of Salamon and Munro's (1967) formula to scale up laboratory size specimen strengths for use in crown pillar design at the Agnes gold mine, South Africa, and was found to give good results that matched observed behaviour quite closely.

Wilson (1972) proposed a pillar formula that took into account the fact that the inner core of a pillar is subjected to triaxial stress conditions, and is surrounded by a yielding zone which contains the inner core. Agapito and Hardy (1982) developed the induced horizontal stress method for oil shale pillars, which is similar to Wilson's method in that it relates both vertical and horizontal stresses developed in a pillar at failure.

Stephansson (1985) described a pillar design procedure, using a boundary element program to determine stresses, at the Kotalahti mine, Finland. Stephansson proposed the equation:

$$\sigma_p = \sigma_o + q_p \sigma_3 \tag{4.7}$$

σ_p = confined pillar strength
σ_o = unconfined pillar strength from equation (4.5)
σ_3 = least principal stress
and where q_p is given by:

$$q_p = \left(\frac{1 + \sin \phi_b}{1 - \sin \phi_b} \right) \tag{4.8}$$

where ϕ_b is the basic friction angle of joints in the pillar.

The first part of this equation, for σ_o, utilises Agapito and Hardy's (1982) scaling relation (equation 4.5), for relating uniaxial compressive strengths of laboratory specimens to *in situ* unconfined compressive strengths. The second part of the equation is an adaptation of Wilson's (1972) confined core method, to take account of the confinement of the core of a pillar. Although Stephansson recognised the need for confinement to be considered when determining the strength of crown pillars, his method is unsuitable. The first part of the equation is developed from data on intact rock, whilst the second part assumes that the pillar contains joints.

The empirical failure criteria given in Murrell (1965) and Hoek and Brown (1980) both take account of the influence of the triaxial state of stress developed in pillars, and represent a more suitable way of determining the bulk strength of a pillar. The Hoek-Brown criterion must be used with care to ensure that the m and s constants are correct. The chart given in Hoek and Brown (1980) of m and s against rock mass quality produces conservative values and gives a failure criterion that underestimates the strength of crown pillars. The updated Hoek-Brown failure criterion parameters (Hoek, 1988) produce results that are less conservative and can be used in pillar design. Figure 3.21 gives a plot of the updated Hoek-Brown failure criterion parameters.

Pillar Load

The state of stress in a pillar is three-dimensional in nature and is dependent on the load applied to the pillar, as well as the location within the pillar that is being considered. The load applied to a pillar is dependent on the pre-mining stress field and the size and location of surrounding openings (Figure 4.7). The stress inside a pillar is dependent on areas of weakness, such as large discontinuities, the proximity of excavations, and the degree of fracturing present in the pillar.

Due to the difficulty in determining the stress distribution within a pillar, mine pillar loading has in the past been evaluated using empirical methods. Bunting (1911) proposed a procedure for pillar design in flat lying, tabular orebodies, that is now identified as the *tributary area method.* In this method, the coverload originally carried by a certain area of rock (the tributary area) is transferred to the pillar when the adjacent rock is mined (Figure 4.8). The pillar is assumed to be uniaxially loaded and the average axial stress for a long rib pillar is given by:

$$\sigma_p = \left(\frac{w_o + w_p}{w_p}\right) p_{zz} \tag{4.9}$$

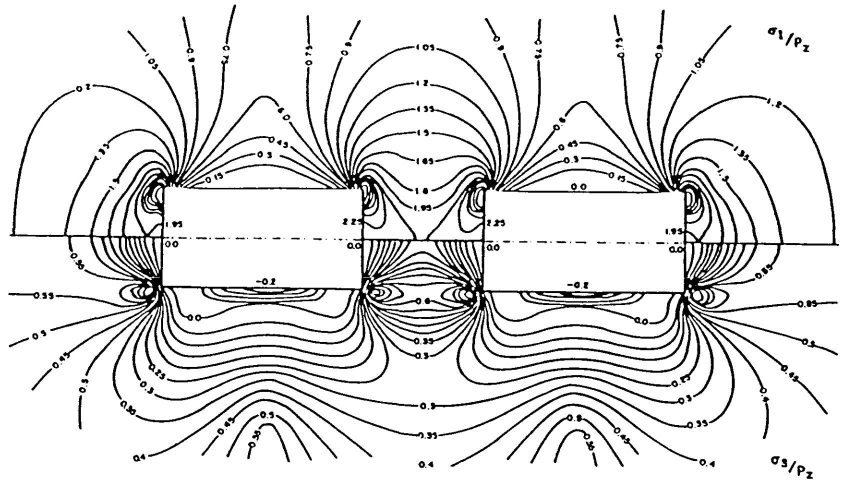
σ_p = average axial pillar stress
p_{zz} = vertical component of pre-mining stress field
w_o and w_p are given in Figure 4.8.

The more general equation for a layout of pillars which are rectangular or square in shape is given by:

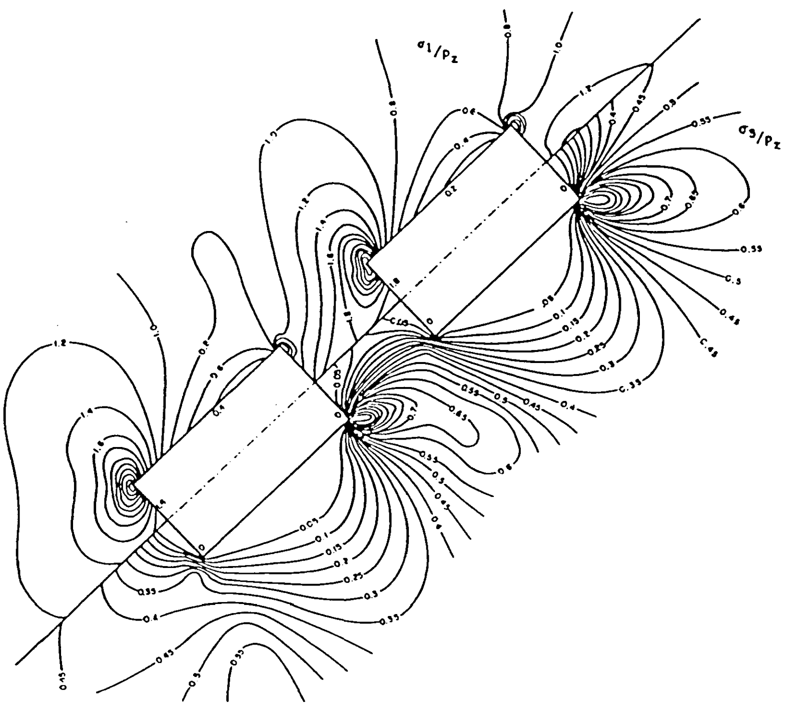
$$\sigma_p = \left(\frac{(a+c)(b+c)}{ab}\right) p_{zz} \tag{4.10}$$

σ_p = average axial pillar stress
p_{zz} = vertical component of pre-mining stress field
a and b are pillar dimensions given in Figure 4.9
c is the stope span.

The attraction of the tributary area method lies in its simplicity, and it predicts pillar behaviour satisfactorily where adequate field data is available for the determination of the shape and volume effects of pillar strength. However, as Brady



Principal stress distributions in the rock surrounding two adjacent excavations aligned normal and parallel to the applied stress direction (K=0.5)



Principal stress distributions in the rock surrounding two adjacent excavations inclined at 45° with respect to the applied stress direction (K=0.5)

Figure 4.7 Stress distributions within pillars at different orientations to the *in situ* stress field (after Hoek and Brown, 1980).

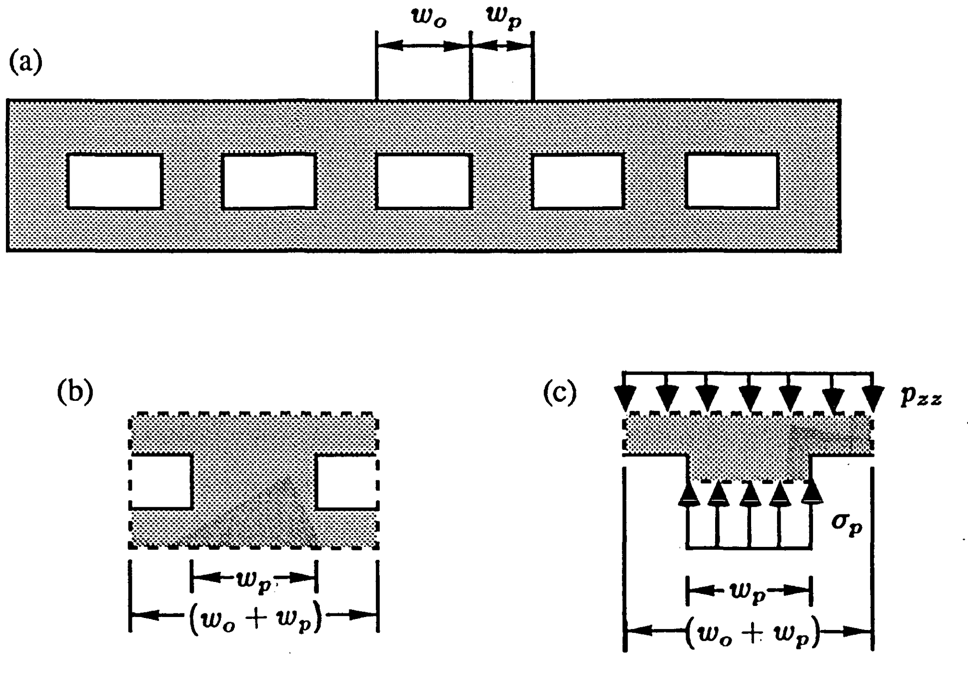


Figure 4.8 Basis of the tributary area method for estimating average axial pillar stress in an extensive mine structure, exploiting long rooms with rib pillars (after Brady and Brown, 1985).

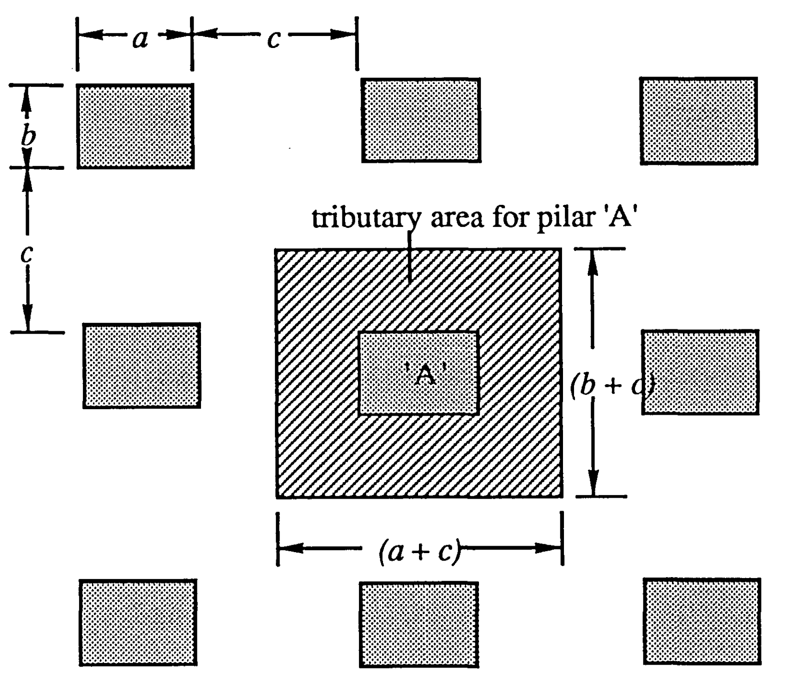


Figure 4.9 Geometry for tributary area analysis of pillars in uniaxial loading (after Brady and Brown, 1985).

(1977) stated, it is only applicable where the number of pillars is large, it disregards the location of the pillar within the mined area, and it takes no account of the field stresses acting in the plane of the orebody.

4.3.3 Numerical Analysis Design

Recognition of these inadequacies in the tributary area method for determining the stress distribution in pillars led to the application of numerical methods in pillar design. In general, quite simple numerical analysis has been employed, but an observational approach (Figure 3.16) has been adopted, whereby monitoring is used to determine the *in situ* rock mass response and provides the feedback necessary to close the design loop. This methodology for pillar design is similar to the conceptual modelling methodology for open stope design (Section 3.4.2). Such an approach is necessary because of the complex nature of pillar behaviour, and the inability to accurately model the stress distributions within pillars. In addition, the quality and quantity of data available for pillar design – *in situ* pillar material properties; location, orientation and condition of discontinuities; post-peak performance of pillars – are generally low.

Numerical analysis design, using a conceptual modelling approach, utilises rock mass classification data to develop a conceptual model of the rock mass, that incorporates the main mechanical features of the rock mass. Elastic continuum behaviour of the rock mass is generally assumed and simple numerical analysis is conducted. Results from the numerical analysis are then compared to actual measured conditions, and the numerical model parameters are adjusted until the predicted behaviour resembles the actual behaviour. The numerical model is then deemed to be calibrated to site conditions, and can be used as a predictive tool for determining pillar dimensions.

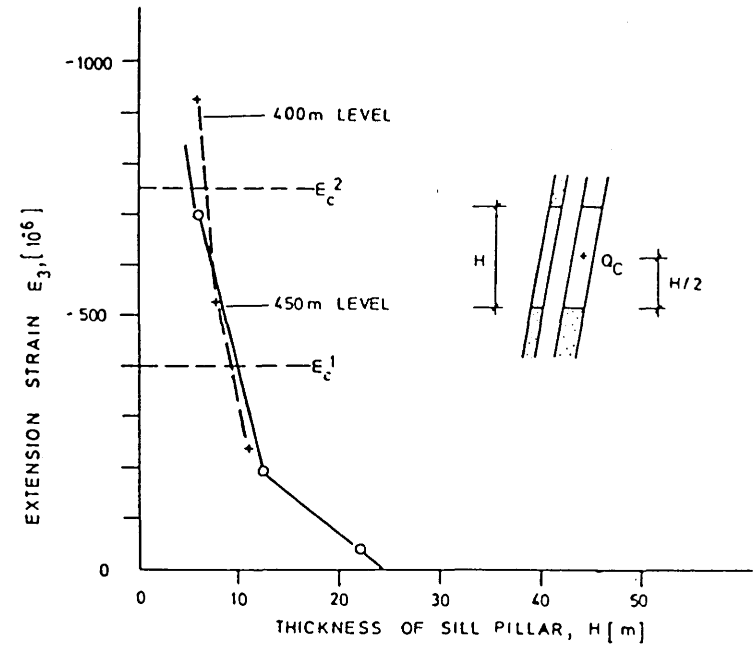
Finite element methods were first applied to the problem of pillar design, and were especially favoured where the number of openings and pillars were limited. Pariseau (1975) and Agapito and Hardy (1982) used finite element methods to model the development of failure zones within pillars. Borg and Krauland (1983) modelled a mining sequence at the Nasliden mine, Sweden, and using an extensional failure criteria due to Stacey (1981), the model was calibrated to mine conditions. This calibrated model was then used to predict stages of failure for open stope and pillar layouts. Borg *et al.* (1984) used a finite element program, again with the failure criterion of Stacey (1981), to dimension the crown pillar at Zinkgruvan mine, Sweden (Figure 4.10).

Finite element methods are preferred by some because they can model complex constitutive behaviour in the near field rock. Coulthard *et al.* (1983) used a two-dimensional finite element program to model a fault passing through a pillar. The finite element program enabled investigation of the effects of sequential excavation of primary stopes and rib pillars, the placement of cemented backfill, and the non-linear behaviour of the rock mass. However, finite element methods have an inherent deficiency, in that they are unable to adequately model complex mine structures in irregularly shaped orebodies because of their need to select unrealistic boundaries to the problem area. Pariseau (1975) in fact, only modelled one pillar because a more complex problem would have required a prohibitive amount of computer resources.

The boundary element method implicitly models infinite boundaries, hence removing the need to define arbitrary problem area boundaries. The method also makes less demand on computer resources than the finite element method. Page and Brennen (1982) used a two-dimensional displacement discontinuity program (Section 2.3) to model a series of stopes and pillars, in order to determine the optimum pillar size. Mathews *et al.* (1983) used a similar program to determine zones of tensile stress within pillars between open stopes. Many workers have adopted the approach, in open stope pillar design, of taking vertical cross-sections of the orebody at the midpoint of the pillar. Where crown pillars are long and continuous this is valid, as plane strain conditions will apply, but care must be exercised when pillars have low length to width ratios along strike. For rib pillars, a horizontal cross-section is usually chosen at the middle of the pillar, with the same care needed as above, concerning plane strain conditions.

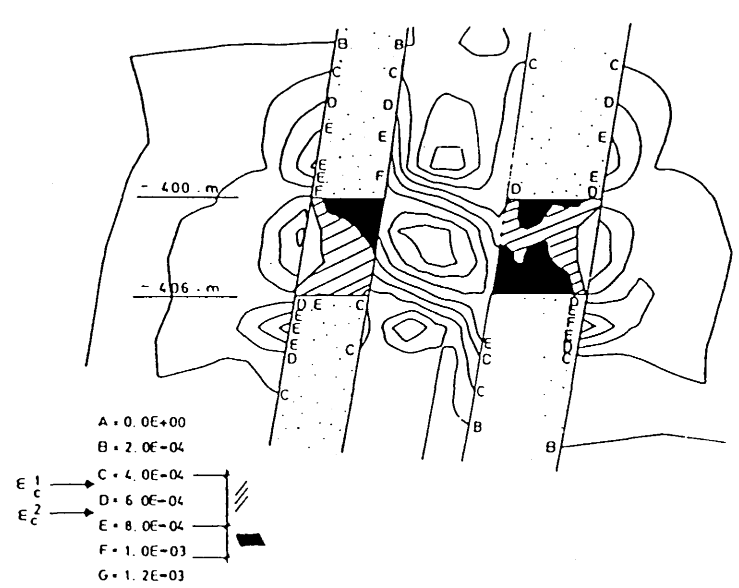
Brady (1977) used a two-dimensional fictitious stress program (Section 2.3) to model the rib pillar between two open stopes at Mount Isa mine, Australia. Brady (1977) developed a failure criterion for the rock mass by back analysis, using the calculated state of stress in the rock when local failure, involving fracture of rock, occurred. This failure criterion did not represent a fundamental mechanical property of the medium, but results were sufficiently consistent to allow acceptance of the pillar design procedure. Lover *et al.* (1983) used a similar boundary element program to dimension a crown pillar at the Kotalahti mine, Finland. Lover *et al.*

(a)



Extension strain versus thickness of sill pillar for mining panels 400m and 450m. Thicknesses corresponding to critical strain ϵ_c^1 and ϵ_c^2 are indicated.

(b)



Strain distribution in the 6m thick sill pillar at 400m level. Notice the development of critical strains ϵ_c^1 and ϵ_c^2 in the main orebody (*right*) and parallel orebody (*left*).

Figure 4.10 Dimensioning of the crown pillar at Zinkgruvan mine, Sweden (after Borg *et al.*, 1984).

(1983) used the failure criteria due to Hoek and Brown (1980) to estimate the triaxial strength of the pillar and determine the factor of safety against failure (defined as the the strength to stress ratio). Stephansson (1985) adopted a similar approach and determined the average factor of safety across a crown pillar, for various pillar widths (Figure 4.11).

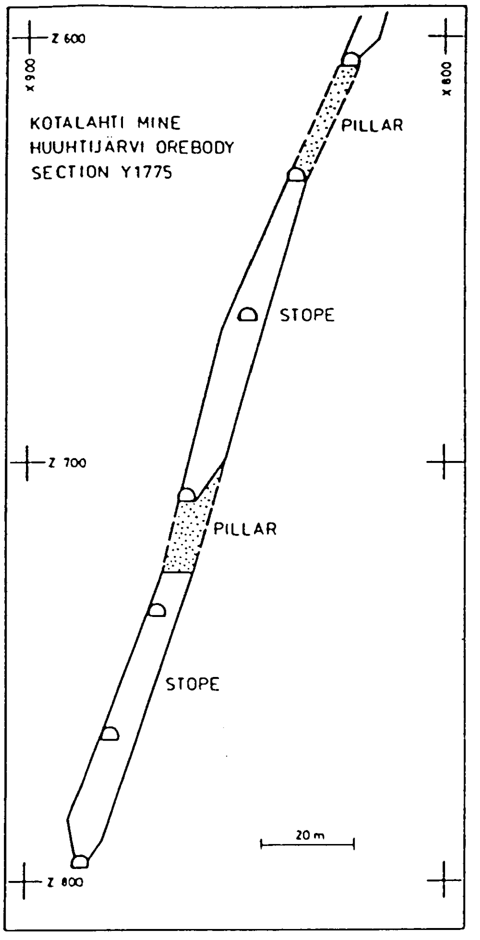
In narrow, steeply dipping, tabular orebodies, three-dimensional modelling using programs such as MINTAB and NFOLD (Section 2.3) is common. A longitudinal section of the orebody, with the complex layout of stopes and pillars, is modelled and then back analysis is conducted using observations of failure or monitoring of stress and/or displacement (Von Kimmelmann *et al.*, 1984; Bywater *et al.*, 1983; Hammett and McKervey, 1985). This type of analysis, using a displacement discontinuity program, is unable to give stress distributions within pillars, but back analysis against existing conditions allows such programs to be used as predictive tools. Watson and Cowling (1985) used a three-dimensional elastic boundary element program for analysing large open stopes at Mount Isa mine, Australia, and were able to model the true three-dimensional behaviour of the rib pillars between the stopes. However, such analysis requires considerable computer resources and is rarely used for general stope design, but is restricted to the design of permanent installations.

4.3.4 Global Stability Design

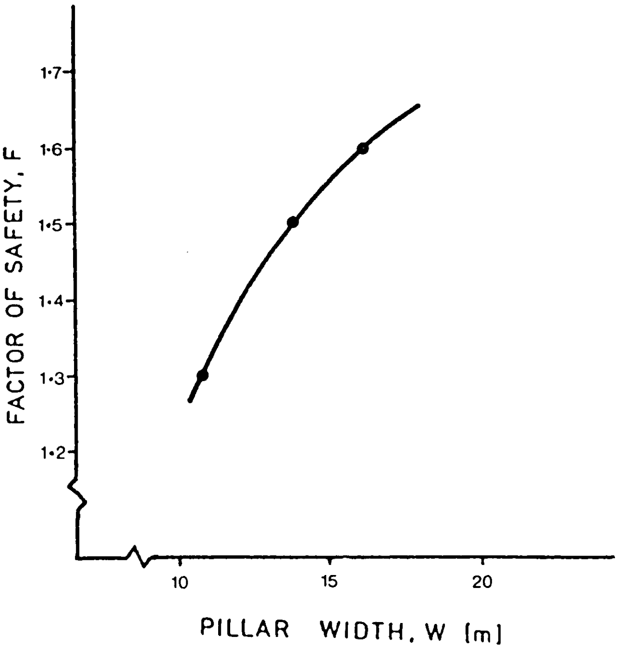
Conventional pillar design is based on the dimensioning of pillars to maintain the intact state of pillar rock and to control displacements of the country rock. Mining inevitably leads to some local failure of pillars and stope walls, but the overriding objective is to ensure that these local failures cannot lead to more extensive failure of the near-field rock. Traditional design methods, as discussed in Section 4.3.2, tend to give conservative results where there are only a few pillars (Hoek and Brown, 1980). These design methods are also mainly aimed at flat lying stratiform orebodies, and are not strictly applicable to steeply dipping, tabular orebodies.

A new approach to the pillar design problem has been the concept of global stability design. The simplest rock stability problem is the loading of a rock specimen in a conventional testing machine, as represented schematically in Figure 4.12.

(a)



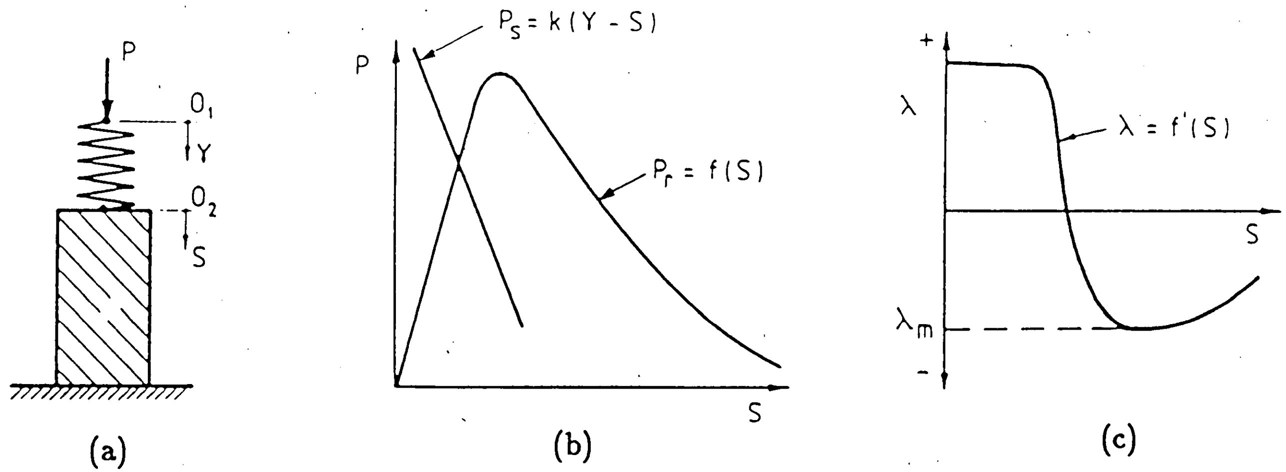
(b)



(a) Cross-section of pillar design at $+710m$, Huutijarvi orebody

(b) Factor of safety versus pillar width at +710, Huutijarvi orebody.

Figure 4.11 Determining factor of safety of a crown pillar at Kotalahti mine, Finland (after Stephansson, 1985).



(a) The country rock represents the spring, and the pillar represents the specimen.

(b) Load–deformation performance characteristic of spring and rock specimen.

(c) Variation of specimen stiffness (λ) with deformation.

Figure 4.12 Analogy between loading a rock specimen in a conventional testing machine and loading a mine pillar by mining induced displacements of the country rock (after Brady and Brown, 1981).

From equilibrium considerations, stable equilibrium of the spring-specimen system is assured if:

$$k + \lambda > 0 \tag{4.11}$$

k = spring stiffness

λ = slope of the specimen force-displacement characteristic defined in Figure 4.12c.

Starfield and Fairhurst (1968) proposed that equation (4.11) be used to establish the stability of individual pillars in stratiform orebodies. Mine pillars are loaded as a result of mining induced displacements of the country rock, which are resisted by the pillar rock. The country rock therefore represents the spring in Figure 4.12a, and the pillar represents the specimen. For a mining layout consisting of several stopes and pillars, the stiffness of pillar i, λ_i, replaces λ in equation (4.11), while the corresponding local stiffness, k_{li}, replaces k. Mine global stability is then assured if for all pillars i,

$$k_{li} + \lambda_i > 0 \tag{4.12}$$

As can be seen in Figure 4.12c, in the elastic range of pillar performance, λ_i is positive, and for elastic performance of the abutting country rock, k_{li} is positive by definition. Pillar instability is liable to occur when λ_i is negative, in the post-peak range, and $|\lambda_i| > k_{li}$.

Brady (1979) and Brady and Brown (1981) described the use of a direct formulation of the boundary element method to determine local stiffness and pillar stiffness in the elastic range. Values of elastic and post-peak pillar stiffnesses for pillars of various width to height ratios were estimated from published data on laboratory specimens and large-scale field tests. They concluded that for massive orebody rock with the same elastic properties as the country rock, any pillar failure may result in instability, whatever the extraction ratio or pillar dimensions. They also concluded that if the orebody rock had a Young's modulus one-third that of the country rock then pillar instability was unlikely. Their conclusions were based on the premise that failure occurred by the generation of new fractures in a pillar. However, the presence of discontinuities has a dominant effect on the post-peak deformation behaviour of the pillar rock. The inference is that natural discontinuities in a rock mass allow slip to occur during the process of development of new fractures in a pillar, and lead to the stable yield of a pillar.

Sarkka (1983) attempted to dimension a crown pillar at the Rautuvaara mine, Finland, using this global stability design method. Sarkka used small scale tests of rock, with the same width to height ratios as in the pillar, to determine the pillar post-peak deformation and stiffness. The results were disappointing, mainly due to the fact that Sarkka had extrapolated the results of laboratory tests on 32 mm diameter cores of intact rock, to the *in situ* behaviour of the pillar. However, the global stability design method does seem to have potential, providing that large-scale testing of rock can be conducted, in order to determine the *in situ* post-peak deformation behaviour of the rock, and providing that numerical modelling is more able to accurately determine the mine local stiffness.

4.4 Adopted Approach to Open Stope Pillar Design

4.4.1 Introduction

Open stope mine pillars are designed to control rock mass displacements throughout the zone of influence of mining, to maintain safe working conditions, and to preserve the condition of unmined ore. In open stope mining without backfill, pillar dimensions are often decided prior to mining starting, with little scope for modification of the final pillar size. Failure of pillars, as a result of inadequate initial dimensioning, can lead to excessive dilution, as well as endangering the regional stability. Over-dimensioning of pillars results in a loss of ore reserve, with the subsequent added expense if the pillar is later mined. Thus, for open stoping without backfill, optimisation of crown pillar dimensions is seen to be of vital importance.

The various pillar design methodologies, that have been described in the literature, were critically assessed in Section 4.3. In practice, the most commonly used design method is empirical design. One of the reasons why numerical analysis is not more widely used is that mine operators have had little success in its practical use. This has not been due to any failure in the numerical methods, but to weaknesses in the design methodology that have resulted in the wrong type of analysis being conducted, or due to insufficient and low quality data being used in the design.

Empirical techniques, however, are not universally applicable, and were developed for particular geological settings. Most empirical pillar strength formulae were developed for shallow, flat lying, tabular coal or oil shale deposits. Little attention has been paid to pillar design in hard rock masses, especially for orebodies that are steeply dipping. Most of the empirical design methods also utilise the tributary area method to determine the pillar loading (Section 4.3.2), which assumes that a pillar is uniaxially loaded. As Figure 4.7 showed, the stress distribution in inclined orebodies is not uniform, and at depth the effect of the minor principal stress on the strength of the rock mass cannot be ignored. Also, at depth, the stress normal to the pillar axis may not be the maximum principal stress. Thus, the empirical techniques can be seen to be inadequate as general design tools because they are only applicable in one geological setting, and because they take no account of the mechanics of pillar failure.

The global stability design methodology (Section 4.3.4) has promise as a pillar design technique, but suffers from the fact that little data on the stiffnesses of hard rock mine pillars exist. For the method to be applied with success, large-scale testing of a rock mass, in order to determine the *in situ* post-peak deformation behaviour of the rock, would be necessary. Such *in situ* rock testing is, however, expensive to undertake.

The numerical analysis design method (Section 4.3.3) suffers from none of the inadequacies described above for the empirical and global design methodologies, and is the adopted design methodology in this thesis for crown pillar design. In the next section an advanced design methodology is developed for the design of crown pillars in narrow, steeply dipping tabular orebodies. This methodology develops a conceptual model of the rock mass, that contains the features of the rock mass that have a direct influence on the mechanics of failure (Section 4.2). Linked to the conceptual models are appropriate numerical modelling programs. These programs should not be used simply in a one run exercise, but the input parameters should be varied until predicted results match field observations, allowing the model to be calibrated to the site conditions. This calibrated model can then be used as a predictive tool for pillar dimensioning.

4.4.2 Advanced Crown Pillar Design Methodology

Model Conceptualisation

The developed crown pillar design methodology is similar in concept to the developed design methodology for open stope design (Section 3.5.2), and is based firmly on an observational approach. A flowchart of the crown pillar design methodology is given in Figure 4.13. As with the open stope design methodology, it is necessary first to develop a conceptual model of the rock mass, in order that the most important aspects of the rock, in terms of the structure and material behaviour, are included. For this conceptualisation the following data are required:

- material properties of the pillar and the surrounding rock mass
- the *in situ* stress field in the vicinity of the pillar
- discontinuity data (dip, dip direction, spacing, condition, *etc.*).

Using these data the behaviour of the pillar can be conceptualised, as to whether it behaves in an intact, jointed or discrete manner. For discrete behaviour, discontinuum analysis of the pillar is necessary if the pillar failure mechanics are to be effectively modelled. For intact and jointed rock behaviour the pillar rock can be approximated as exhibiting continuum or pseudo-continuum behaviour, and to be amenable to elastic analysis.

Failure Criterion For Crown Pillar

The *in situ* strength of a crown pillar is best defined using the empirical equation given in Hoek and Brown (1980), as the influence of the triaxial state of stress is taken into account. Hoek and Brown (1980) developed the following equation:

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^2} \tag{4.13}$$

σ_1 = major principal stress at failure

σ_3 = minor principal stress

σ_c = uniaxial compressive strength of the rock mass

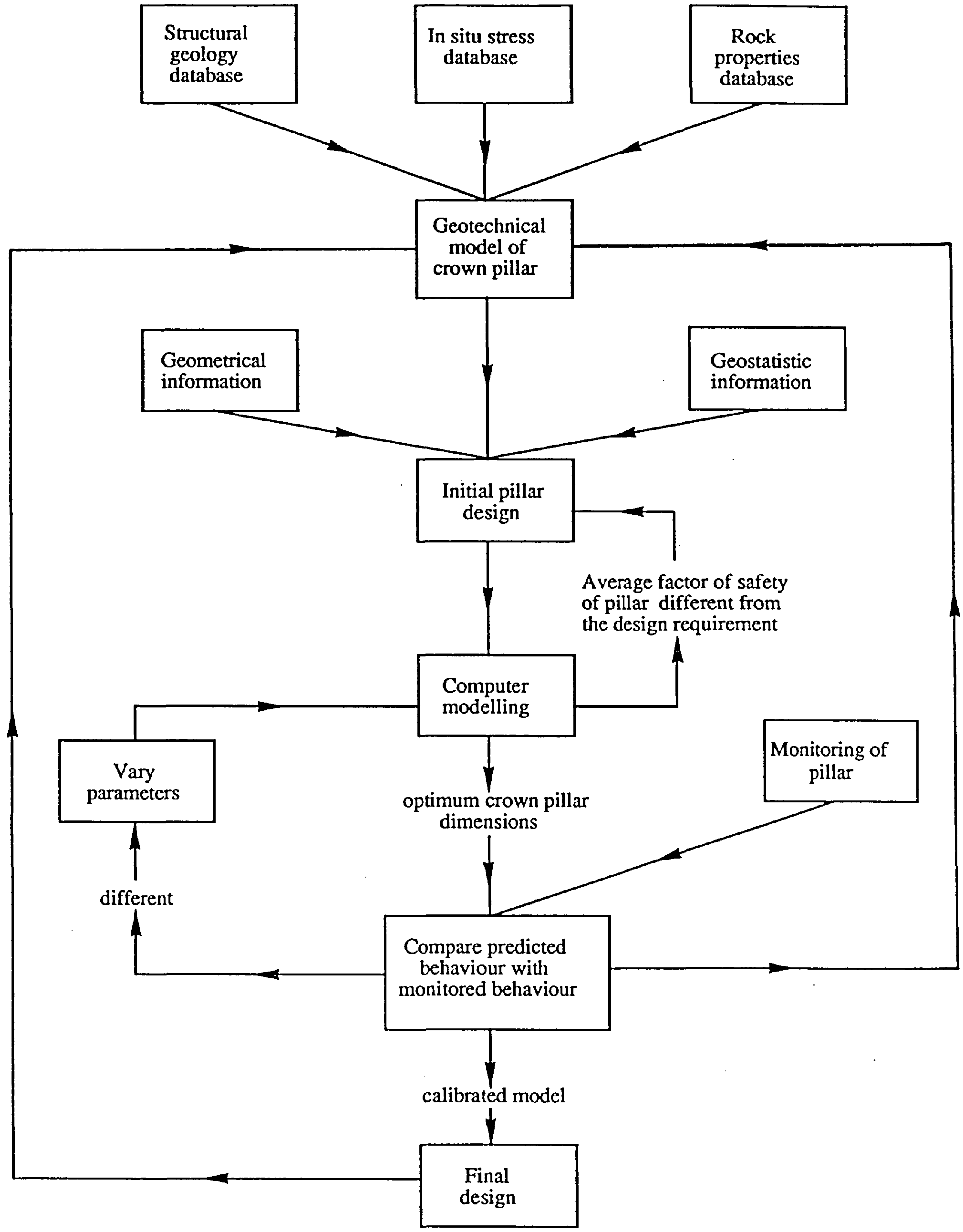


Figure 4.13 Flowchart of the developed crown pillar design methodology.

The m and s constants for equation (4.13) can be determined using equations that relate rock mass rating (RMR') with m and s. The uniaxial compressive strength of the rock mass is derived from the uniaxial compressive strength of intact rock using the equation:

$$\sigma_c(rock\ mass) = \sqrt{s} \cdot \sigma_c(intact\ rock) \tag{4.13b}$$

The original equations relating RMR' with m and s, given in Hoek and Brown (1980), were found to be too conservative for crown pillar design. Hoek (1988) presented a new set of equations for undisturbed or interlocking rock masses:

$$\frac{m}{m_i} = \exp\left(\frac{RMR' - 100}{28}\right) \tag{4.14}$$

$$s = \exp\left(\frac{RMR' - 100}{9}\right) \tag{4.15}$$

m_i	=	value of m for intact rock
RMR'	=	modified version of Rock Mass Rating

RMR' is determined using the CSIR rock mass classification scheme due to Bieniawski (1976) and shown in Table 3.3. However, RMR' ignores the adjustment factor for joint orientation, as these values have been found to not apply to crown pillars. Also, the groundwater factor is set to 10, equivalent to completely dry conditions, as the influence of groundwater pressure should be taken into account by any analysis.

By determining a RMR', and using equations (4.14) and (4.15), a failure criterion for a crown pillar can be developed. This failure criterion is only used as a first estimate of rock strength, and the *in situ* rock strength can be found by back analysis, comparing monitored and modelled crown pillar behaviour.

Initial Layout

Stope and pillar layout is a subjective process, as there are many conflicting design issues. For crown pillar design in open stoping without backfill, the pillars should be as small as possible because they are in ore, and they should be located in low grade ore if possible. At the same time pillars have a support role and they need to be sufficiently large and well located to perform effectively. Pillars should also be located where there are inferred to be no adversely orientated structural features. A pillar that is intended to provide intact, elastic resistance to country rock displacements needs to be located where slip cannot develop in its interior. In orebodies that occur closely together and parallel to one another, pillar axes should be aligned. Pillars whose axes are offset may result in a stress distribution that imposes excessive shear stress on planes of weakness orientated parallel to the orebodies. Figure 3.22 shows the layout of stopes and pillars in two such orebodies at Mount Isa mine Australia, where it can be seen that pillars have been aligned.

Design Analysis

Once a failure criterion for a pillar has been determined and the layout of the stopes and pillars decided, then the next stage of the design methodology is the numerical modelling of the stope and pillar layout. If the rock mass can be approximated to having elastic continuum behaviour then simple boundary element analysis is appropriate to determine the stress and displacement distributions.

The most suitable program to use is dictated to a large degree by the design problem itself. In Section 2.3.5, a suite of boundary element programs were assessed, and particular attention was given to their applicability to pillar design. It was shown that two-dimensional displacement discontinuity programs are of little use for pillar design, as they are unable to model the stress distribution within pillars, because the method implicitly assumes that the orebody has zero thickness. However, three-dimensional displacement discontinuity programs, such as MINTAB, are of use in crown pillar analysis (Section 4.3.3). MINTAB cannot model the stress distribution within pillars, but it can model the complex layout of stopes and pillars commonly found in tabular orebodies, and can give stresses acting perpendicular to an orebody. Back analysis using observation of failure or monitoring of stresses and/or displacements can then be used to determine optimum stope sizes, pillar sizes and mining sequences.

Three-dimensional analysis of crown pillars is required where the layout of stopes and pillars in an orebody is such that plane strain conditions cannot be assumed at the cross-section of the pillar being considered. Hocking (1978) concluded that two-dimensional elastic plane strain analysis approximated the correct three-dimensional solution to 90-95%, when the cross-section for the two-dimensional analysis was located at a distance of two excavation 'diameters' away from a face or intersection. For a crown pillar in a narrow, tabular orebody this means that plane strain conditions approximately exist if the cross-section of the pillar is taken such that it is at a distance of twice the true dip length of the stope hangingwall away from the stope end. If the stopes above and below the crown pillar are different sizes then the larger hangingwall dip length should be used.

If two-dimensional analysis can be used, then the boundary element programs BEM11 and TWOFS were identified as being of use in crown pillar design (Section 2.3.5). These programs were not designed for use in narrow orebodies, and hence care must be taken in discretising the boundaries of the stopes above and below the crown pillar. The element lengths should be less than the orebody width, otherwise problems can occur in the convergence of the solution. The difference between BEM11 and TWOFS is that BEM11 is a complete plane strain program, whereas TWOFS is a conventional plane strain program. Conventional plane strain analysis assumes that a principal stress is parallel to the long axis of the excavation, *i.e.*, perpendicular to the two-dimensional cross-section. Brady (1979) found that if the long axis of the excavation lay at an angle greater than 20° to a principal stress then complete plane strain analysis was necessary.

The two-dimensional boundary element programs can be used to determine the stress and displacement distributions within a crown pillar. When used with the Hoek-Brown failure criterion, factors of safety against failure can also be determined. The factor of safety is defined as the ratio of the major principal stress, σ_1, calculated using the failure criterion (equation 4.13), to σ_1 calculated by numerical analysis. The average factor of safety for a crown pillar is taken by averaging the factors of safety across the centre of the pillar, that is from stope to stope along the centre of the pillar. An average factor of safety below 1 is interpreted as failure. An average factor of safety between 1 and 1.3 is interpreted as temporarily stable. An average factor of safety between 1.3 and 1.6 is interpreted as stable, and suitable for crown pillars that are designed to be semi-permanent. An average factor of safety greater than 1.6 should be used for pillars which are required to remain stable and not fail. Using these factors of safety a preliminary value can be set for the optimum crown pillar size. This may be altered later, after retrospective analysis using monitoring data.

Rock Performance Monitoring

Once an initial design analysis of a crown pillar has been conducted and the pillar size determined, then monitoring of the pillar has to be considered. Monitoring of pillars during mining, as Worotnicki *et al.* (1980) demonstrated, is essential if the behaviour of a pillar to increased levels of stress is to be understood. Monitoring of stress changes in crown pillars is usually achieved using overcoring methods or vibrating wire stressmeters, and displacements are usually measured using rod or wire extensometers. Further discussions on monitoring programs of pillars are given in Bywater *et al.* (1983); Bawden and Milne (1987); Worotnicki *et al.* (1980); Lover *et al.* (1983).

Retrospective Analysis

Using the monitoring information, together with visual observations of pillar performance, retrospective analysis of a crown pillar design can be undertaken. Extensometer measurements can reveal whether fractures are developing, whether pre-existing fractures are opening up in the pillar and whether slip is occurring. If large scale slip is found to occur along a particular joint set, then it is important that this joint set is specifically modelled in the analysis, as it will have a controlling influence on the stress distribution within the pillar and on the pillar failure mechanism. Stress measurements in pillars are also of use in identifying unexpected behaviour in pillars as well as for calibrating stress analyses.

In the developed design methodology, monitoring of a pillar is used to determine whether the conceptual model of the pillar has to be modified, to take account of any newly revealed rock mass behaviour. Monitoring also reveals inaccuracies in input parameters to the numerical models. By modifying the input parameters and conducting the analysis again, the model will eventually be calibrated to the site conditions. This retrospective analysis can be seen as the feedback that closes the design loop in Figure 4.13.

As mining progresses, and more data on the actual crown pillar performance become available, then the numerical model can be calibrated. The model can then be used to design other crown pillars in the same orebody. However, if the rock conditions change, then the model has to be re-evaluated, which could mean re-analysis using a different numerical analysis program, or simply re-calibration of the model, depending on how much the rock conditions had changed.

CHAPTER 5
KNOWLEDGE-BASED SYSTEMS

5.1 Introduction

Artificial Intelligence (AI) evolved as a branch of computer science, paralleling other branches of computer science such as languages, data structures, operating systems and numerical algorithms. Research in AI has several goals (Duda and Shortliffe, 1983). One is the development of computational models of intelligent behaviour, including both its cognitive and perceptual aspects. A more engineering orientated goal is the development of computer programs that can solve problems normally thought to require human intelligence.

Early work in AI was based on the the fact that computers are not just calculators, but can manipulate symbols. Work concentrated on finding simple and powerful reasoning techniques that could be applied to many different problems. Newell *et al.* (1963) developed a general purpose problem solver, GPS, that could prove theorems and solve puzzles and a wide variety of logical problems. However, attempts to apply these general methods to larger problems were mostly unsuccessful. These general problem-independent heuristic methods were incapable of handling the combinatorial complexity that was encountered.

Researchers then started to look at how humans solved problems. They found that they rarely solved problems from first principles, as their computer programs had been trying to do. They found that people possess knowledge which their computer was ignorant about. This knowledge was of many kinds and was used in many different ways; to clarify a problem, to suggest approaches to take, judging the reliability of facts, and deciding whether a solution is reasonable. This recognition that knowledge was as important as reasoning led to a fundamental shift in AI research. The first generation knowledge-based systems emerged when problem solving strategies were combined with domain specific knowledge of a specialised area. These programs include the DENDRAL program for mass-spectrum analysis (Lindsay *et al.*, 1980), the MYCIN program for infectious disease diagnosis (Shortliffe, 1976), the PROSPECTOR program for mineral exploration (Duda *et* *al.*; 1979) and the R1 program for configuring VAX computer systems (McDermott, 1980).

This chapter describes the new technology of knowledge-based systems and explains what they are, how they work, and how they differ from conventional programs. The actual process of developing a knowledge-based system, from task selection, through knowledge collection, to the programming of the system, is described in detail in Section 5.4. The different types of knowledge-based system are discussed in Section 5.5, and within each type examples of working systems, with an emphasis on those for the earth sciences, are given. The technology of knowledge-based systems is shown to be potentially of great use in all aspects of engineering rock mechanics. In particular, the application area of intelligent modelling tools (Section 5.5.2) is highlighted and in Chapter 6 a prototype of such a tool for crown pillar design using numerical analysis is developed. The Glossary contains a short list of terms used in the knowledge-based systems literature.

5.2 Components of a Knowledge-Based System

5.2.1 Introduction

There are three fundamental elements in any knowledge-based system. In this thesis they are referred to as the knowledge base, the inference engine and the working memory. A schematic diagram of a typical knowledge-based system is given in Figure 5.1. The knowledge base is where the domain specific knowledge resides. The knowledge is kept totally separate from the reasoning mechanism, which is contained in the inference engine. In normal operation the inference engine employs the information contained in the knowledge base to interpret the current contextual data in the working memory. Ideally, everything that is application dependent is kept in the knowledge base so that the inference engine can be a multi-application tool.

5.2.2 Knowledge Representation

Work in AI has shown that expertise in a task domain requires substantial knowledge about a domain. The effective representation of domain knowledge is therefore generally considered to be the keystone to the success of AI programs (Fikes and Kehler, 1985). Domain knowledge typically has many forms, including descrip-

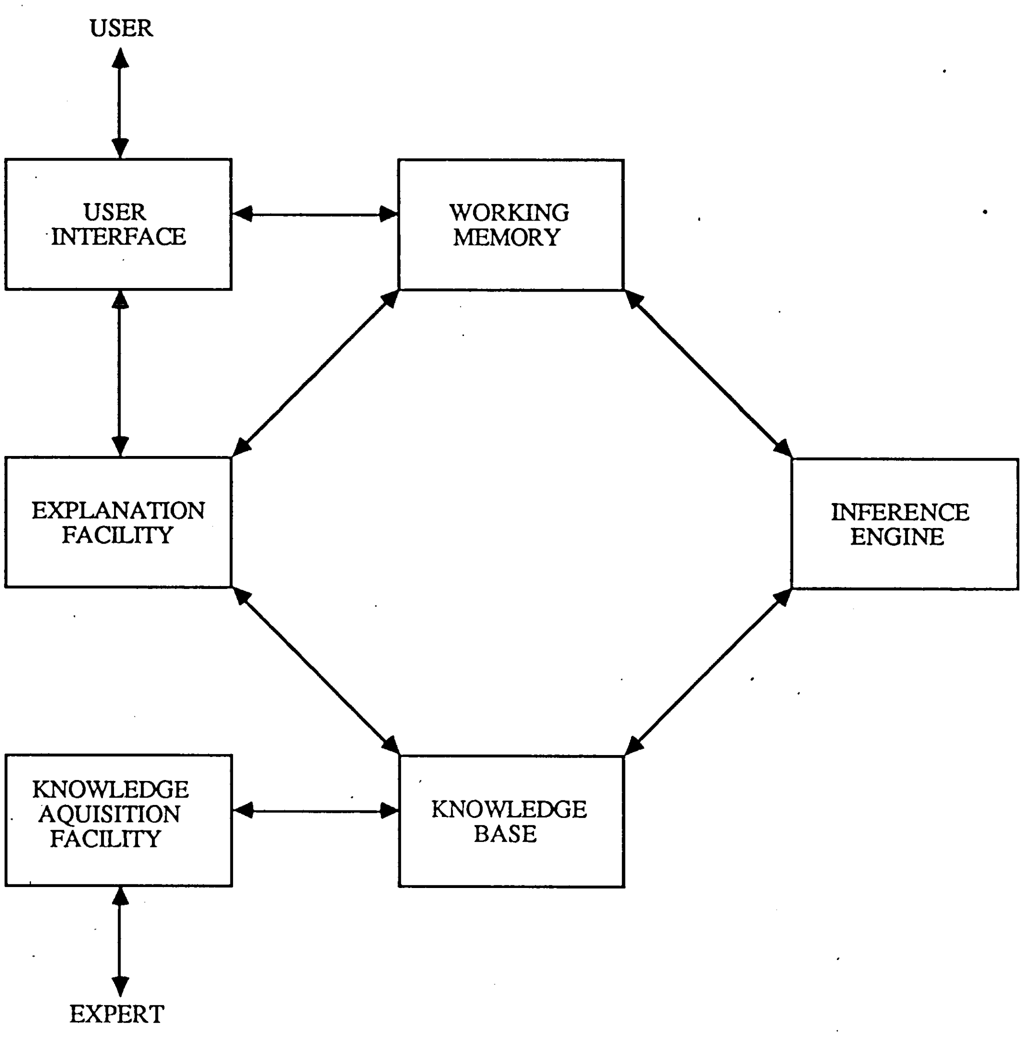


Figure 5.1 Architecture of an expert system (after Maher, 1987).

tive definition of domain-specific terms, descriptions of individual domain objects and their relationships to each other, and criteria for making decisions. Various methods of representing this knowledge have been developed, the most common being rules, frames, and semantic networks.

Rule-based systems were originally used in cognitive modelling of short-term memory (Michaelson *et al.*, 1985). Since knowledge-based systems attempt to imitate people, it was natural that rules would be used. At present, production rules are by far the most common means of representing knowledge. Rules consist of a set of conditions, and a set of actions (often called IF...THEN rules). If all of the conditions in a rule are true, then the actions are executed. The conditions are contained in the 'IF' part of the rule, and the actions in the 'THEN' part. Figure 5.2 shows typical rules from some knowledge-based systems.

To represent more complex knowledge structures, a number of representation schemes have been developed, which are often referred to as slot-and-filler structures (Mackerle and Orsborn, 1988). They share the notion that complex entities can be represented as a collection of attributes and associated values. One such representation is semantic networks. Semantic networks can describe both objects and events. Information is represented as sets of nodes connected to each other by means of labelled arcs. Nodes represent objects and descriptors, and the arcs represent relations between the nodes. Figure 5.3 gives an example of a semantic network.

Another general scheme is frames, usually used to represent complex objects. A frame consists of a collection of slots that describe aspects of the objects (Figure 5.4 shows an example). Slots may contain values, default values or pointers to other frames describing other objects. Also, with each slot may be associated a set of preconditions that must be met by any filler. Procedural information can also be associated with a slot, for instance to determine the value for the slot. This technique is called procedural attachment, and related frames can be grouped together to form a frame system. Frame systems also provide a system to inherit attributes from a taxonomy of entities.

5.2.3 Knowledge Reasoning

The inference mechanism in a knowledge-based system is the part of the system that contains the knowledge reasoning strategy. Various strategies have been

(a) The R1 system (configuring VAX systems):

IF: (1) The current context is assigning devices to Unibus modules, and
(2) There is an unassigned dual-port disk drive, and
(3) The type of controller it requires is unknown, and
(4) There are two such controllers, neither of which has any devices assigned to it, and
(5) The number of devices that these controllers can support is known

THEN: (1) Assign the disk drive to each of the controllers, and
(2) Note that the two controllers have been associated and that each supports one device.

(b) The MYCIN system (medical diagnosis):

IF: (1) The site of the culture is blood, and
(2) The identity of the organism is not known with certainty, and
(3) The stain of the organism is gramneg, and
(4) The morphology of the organism is rod, and
(5) The patient has been seriously burned

THEN: There is weakly suggestive evidence (0.4) that the identity of the organism is pseudomonas.

(c) The PROSPECTOR system (mineral exploration):

IF: There is hornblende pervasively altered to biotite

THEN: There is strong evidence (320, 0.001) for potassic zone alteration.

Explanation

(a) Rule that the R1 system uses to configure DEC's VAX systems.

(b) Rule that MYCIN uses to perform medical diagnosis. The number 0.4 indicates the degree to which the conclusion follows from the evidence on a scale of 0 to 1.

(c) Rule used by PROSPECTOR in mineral exploration. The number 320 indicates how sufficient the evidence is for establishing the hypothesis if the evidence is, in fact, present; the number 0.001 indicates the degree to which the absence of this evidence will rule out the hypothesis. Both these numbers are multipliers. Values greater than 1 increase the likelihood of the sufficiency or necessity of the evidence for establishing the hypothesis, and values less than 1 decrease the likelihood.

Figure 5.2 Sample IF–THEN rules from three knowledge-based systems (after Duda and Gaschnig, 1981).

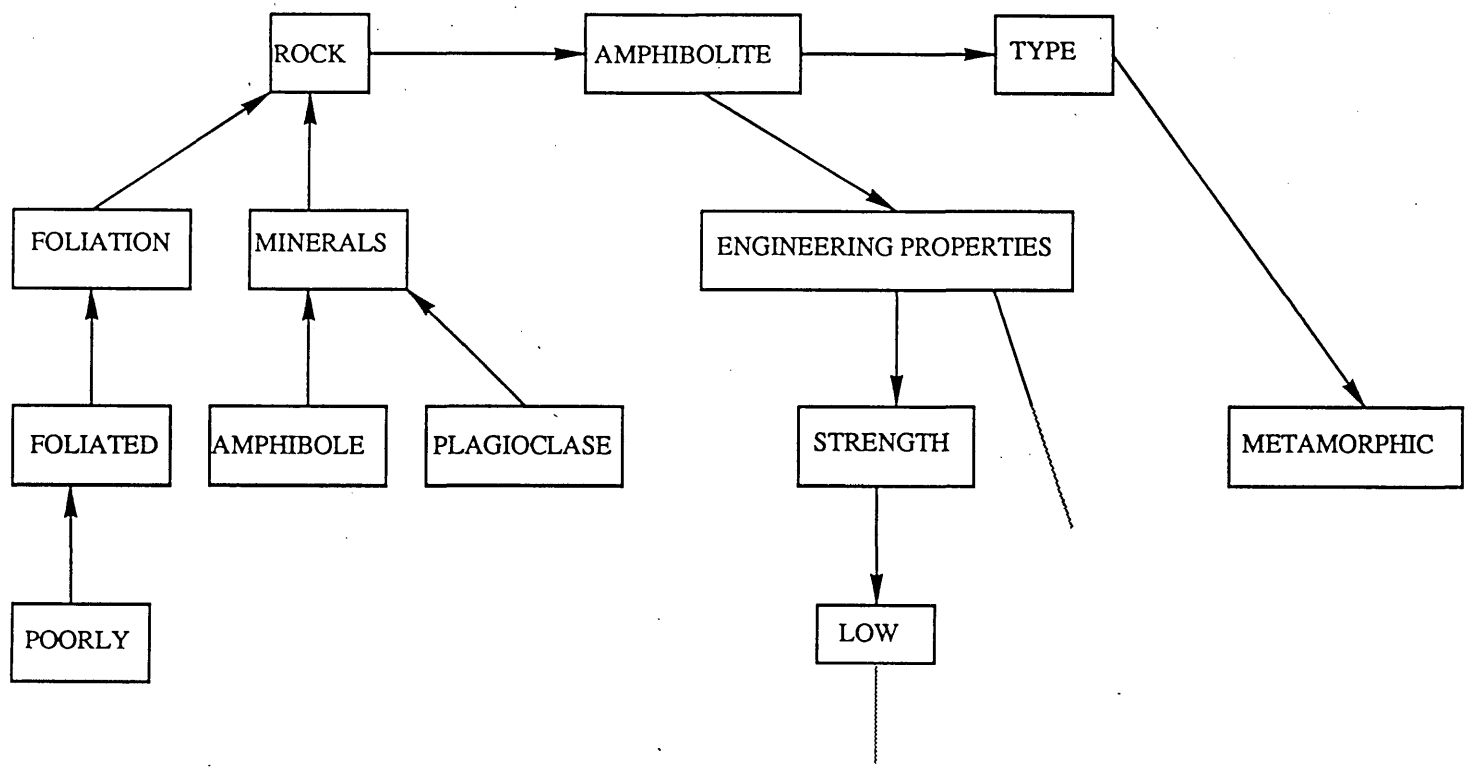


Figure 5.3 Schematic semantic network representation (after Dershowitz and Einstein, 1984).

```
Frame : TRUCKS in Knowledge Base TRANSPORTATION
    SuperClases : VEHICLES
    SubClasses : BIG.NON.RED.TRUCKS, HUGE.GREY.TRUCKS
    MemberOf : CLASSES.OF.PHYSICAL.OBJECTS

                .
                .
                .
MemberSlot      : HEIGHT from PHYSICAL.OBJECTS
    ValueClass  : INTEGER
    Cardinality : Min : 1
    Cardinality : Max : 1
    Unit        : INCHES
    Comment     : " Height in inches "
    Values      : Unknown
                .
                .
                .
                .
                .
OwnSlot         : LONGEST from CLASS.OF.PHYSICAL.OBJECTS
    ValueClass  : TRUCKS
    Cardinality : Min : 1
    Cardinality : Max : 1
    Comment     : " The tallest known truck "
    Value       : Unknown
                .
                .
                .
                .
```

This frame describes class TRUCKS as a subclass of class VEHICLES and as a member of class CLASSES.OF.PHYSICAL.OBJECTS. MemberSlots in the TRUCK frame like HEIGHT provide a prototype description of every class member. OwnSlots like LONGEST describe attributes of the class as a whole.

Figure 5.4 Frame representation (after Fikes and Kehler, 1985).

```
Unit      : TRUCK1 in Knowledge Base TRANSPORTATION
Member    : THINGS.OWNED.BY.PAUL, HUGE.GREY.TRUCKS

            .
            .
            .
OwnSlot         : OWNER
    ValueClass  : MEN (UNION DOCTORS LAWYERS)
                      (NOT.ONE.OF FRED)
                      AGENTS
    Cardinality.Max : 1
    Values      : PAUL
                .
                .
                .
OwnSlot         : Wheels from HUGE.GREY.TRUCKS
    Cardinality.Min : 16
    Comment     : " The vehicles wheels "
    Value       : Unknown
```

This frame, which represents a truck, specifies (by means of the Cardinality.Min part of the WHEEL slot) that the truck must have at least 16 wheels and (by means of the ValueClass part of the OWNER slot) that the owner must be a man who is either a doctor or lawyer and not FRED.

A frame such as TRUCK1 can either have a member link to one or more class frames (e.g. to represent that TRUCK1 is a member of both TRUCKS and THINGS.OWNED.BY.PAUL). A frame is said to inherit the member slots of the class frames to which it has member links. For example, the TRUCK1 frame would acquire the OwnSlot, Height from the frame for TRUCKS (see Figure 5.4).

Figure 5.5 A frame exhibiting inheritance (after Fikes and Kehler, 1985).

adopted in the past and are described below. Most rule-based programs are production systems in which matching and scheduling are explicitly defined by the inference engine. Production systems are either consequent or antecedent driven. Frames allow the implementation of deeper level reasoning such as abstraction and analogy – important expert activities. The development of domain independent reasoning strategies has led to the production of expert system *shells*, which are pieces of software that contain the ability to capture knowledge in rules or frames, and control strategies to manipulate the knowledge (Section 5.4.4).

Forward-chaining systems (antecedent or data driven), consist of a continuous sequence of cycles terminating when a rule's action dictates a halt. At each cycle, the system scans the antecedents (IF parts) and determines all rules with antecedents that are satisfied by the contents of the database. If there is more than one rule, then one is selected by means of a conflict resolution strategy, which is commonly the order of the rules in the knowledge base. All actions associated with the selected rule are then performed and the database is updated. Forward chaining, hence, essentially consists of putting the rules in a queue and then using a recognise-and-act cycle on them.

Backward chaining (consequent or goal driven) strategies use rule consequents, which represent goals, to guide the search for rules to fire (execute). The system collects the rules that can satisfy the goal in question. To satisfy each antecedent, which represents a sub-goal, the system collects those rules whose consequents satisfy its value. The process works backwards through the rules from consequents to antecedents, in search of a casual chain that will satisfy the goal.

The control of frame systems is usually much more involved than that for the production rule systems which use forward or backward chaining. Frame-based representation extends the systems' explicit facts by automatically performing inferences as part of its assertion and retrieval operation. One of the most important inference methods is what is known as *inheritance*. The use of member links augments the descriptive information in a frame. Any frame can have a member link to one or more class frames and a frame is said to inherit the member slots of the class of frames to which it has member links. These inherited slots become own slots of the member frame, since they represent attributes of the member slot itself. Figures 5.5 shows an example of a frame with inheritance.

5.3 The Difference Between Knowledge-Based Systems and Conventional Programs

A good standard definition of knowledge-based systems is given by Gaschnig *et al.* (1981):

'*Knowledge-Based Expert Systems are interactive computer programs incorporating judgement, experience, rules-of-thumb, intuition, and other expertise to provide knowledgeable advice about a variety of tasks.*'

It is often argued that existing conventional programs already do this. Waterman (1986) stated that the most basic difference between knowledge-based systems and conventional programs is that knowledge-based systems manipulate knowledge, whilst conventional programs manipulate data. Adeli (1985) listed six main differences between knowledge-based systems and conventional programs.

- Knowledge-based systems are knowledge intensive programs.
- In knowledge-based systems, knowledge is usually divided into many separate rules.
- The rules forming the knowledge base are separated from the methods of applying the knowledge to the current problem.
- Knowledge-based systems are highly interactive.
- Knowledge-based systems have user-friendly / intelligent user interfaces.
- Knowledge-based systems, to some extent, mimic the decision making and reasoning process of human experts. They can provide advice, answer questions and justify conclusions.

Increasingly, engineering software is becoming interactive, and programs have always contained knowledge in the form of limitations, assumptions and approximations. Fenves (1986a) gave a better characterisation:

'*An algorithmic program uses a small amount of knowledge (e.g. the knowledge of matrix multiplication) repeatedly over many cycles, whereas knowledge-based systems typically have to search a large amount of knowledge at each cycle, and a particular piece of knowledge may only apply once.*'

There are essentially four basic features that separate knowledge-based systems from algorithmic programs (Fenves, 1986a).

1. Separation of Knowledge Base and Control

The knowledge base and the inference mechanism are totally separate. All knowledge is contained within the knowledge base and the problem solving formalisms appear in the inference engine, which 'executes' the knowledge base.

2. Transparency of Dialogue

There is some form of explanation facility to convey to the user the inference process actually used. This does not include traditional help facilities. The explanation facility allows the line of reasoning and the order in which the rules were executed to be traced. The explanation facility also allows the rule that is being tested and the required facts to be viewed, along with the current value of any facts or conclusions.

3. Transparency of Knowledge Representation

The domain dependent knowledge is incorporated in the program in a readable and understandable fashion.

4. Incremental Growth Facility

The knowledge-based system can be used with a subset of its ultimate knowledge base, and its knowledge base can be incrementally extended over a period of time without major restructuring of the knowledge base.

Figure 5.6 shows the characteristics exhibited by knowledge-based systems – expertise, symbolic reasoning, depth and self-knowledge. These are discussed in more detail below.

Expertise

Knowledge-based systems must perform well, and if they are to be considered as expert systems they must achieve the same level of performance as an expert in the domain of interest. In this respect, the solutions need to be reached quickly as well as being good solutions, and the programs need to be able to take shortcuts

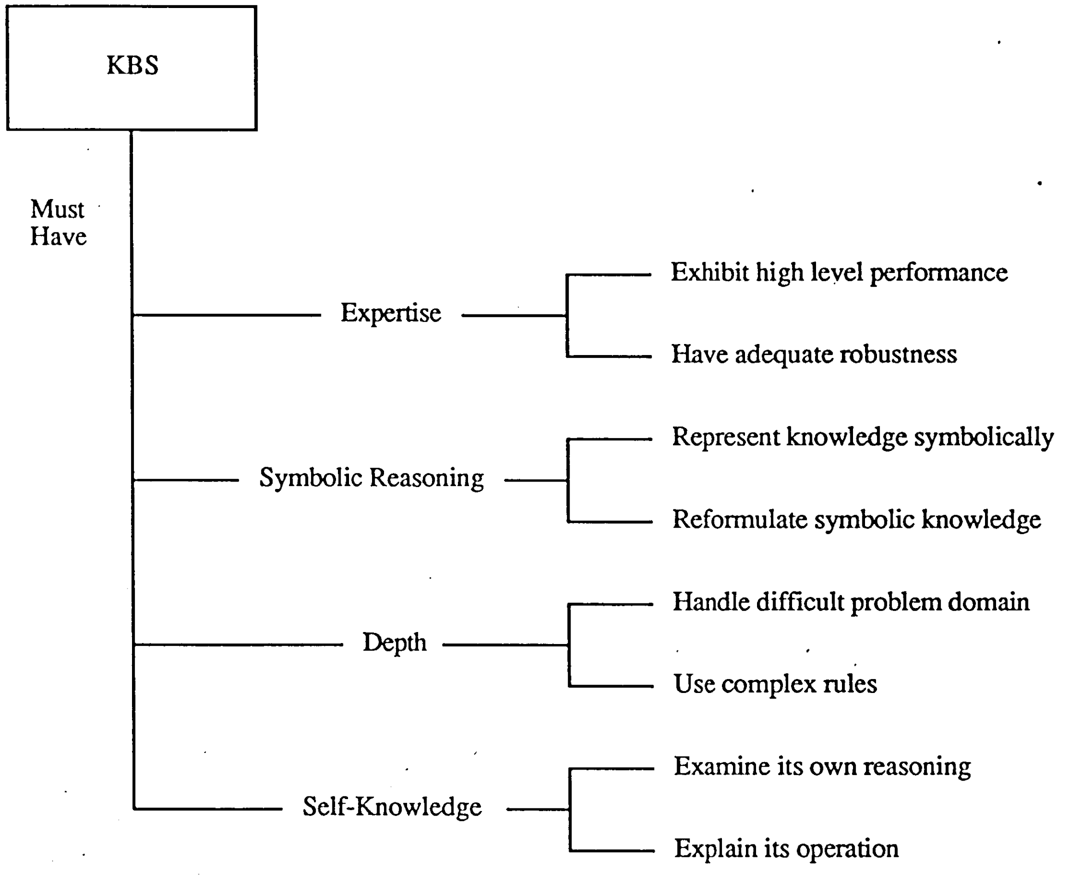


Figure 5.6 Characteristics of a knowledge-based system (after Waterman, 1986).

using heuristics that experts possess. Knowledge-based systems also need to be robust, *i.e.*, they need to have a broad understanding of the subject domain, and to be able to reason from first principles if necessary. Knowledge-based systems should also be able to deal with incorrect or missing data.

Symbolic reasoning

Human experts solve problems by representing problem concepts as symbols and apply various strategies and heuristics to manipulate these concepts. A heuristic is defined as the study or practice of procedures that are valuable but are incapable of proof.

Depth

A knowledge-based system must have depth – that is it should operate effectively in a narrow domain containing a difficult problem. Thus the rules will be many and will be complicated, such that a real world problem can be tackled.

Self-knowledge

A knowledge-based system has knowledge that lets it reason about its own operation. The knowledge that a system has about how it reasons is called *meta-knowledge*, which means knowledge about knowledge. Most knowledge-based systems have an explanation facility which allows the user to see how the system reached its conclusion. This self-knowledge is important because it allows the user to check the reasoning process, which helps speed up the debugging of the system during development. Self-knowledge also allows the user to change input parameters, and to observe how this effects the system operation.

5.4 Development of a Knowledge-Based System

5.4.1 Introduction

Conventional philosophy, as stated by Hayes-Roth *et al.* (1983), is based on the assumption that a knowledge-based system is generated through the co-operation of two groups of individuals: one or more domain practitioners with experience and knowledge about the application domain, and one or more knowledge engineers who are knowledgeable about several knowledge-based system frameworks so that they can:

- choose an appropriate representation
- guide the domain practitioners in developing the relevant knowledge base
- implement the domain knowledge in the selected framework.

Increasingly, however, knowledge-based systems are being developed by application programmers, educated in their respective application domains, and able to incorporate substantial amounts of domain knowledge into the knowledge-based system, based on their personal experience (Fenves, 1986a). The reason for this is that there are so few knowledge engineers and explaining domain knowledge is probably harder than explaining it to a program (Fenves, 1986a). Some of the issues concerned with the implementation of knowledge-based systems are discussed below.

5.4.2 Task Selection

Careful selection of the task to be encapsulated in the knowledge-based system is essential for the success of the system. The task must be clearly defined, fairly narrow in scope, and be a task that a practitioner performs reasonably often and within a reasonable period of time. A 'reasonable' length of time is hard to define, but it is generally understood that the task should only take a few hours, rather than weeks. However this is too simplistic, as some tasks take weeks but contain a lot of repetitive work and still can be successfully modelled using a knowledge-based system.

The main criteria is the complexity of the task and the degree to which it involves creative thought. If a task requires days or weeks of creative thought by a practitioner, then it is likely that it is not suitable for a knowledge-based system. The task should also be rich in reasoning, otherwise an algorithmic solution is probably more appropriate. In order to test the system there should be a substantial library of case studies or test cases. From a management point of view the task area must have a clear, and economic value for the organisation building the knowledge-based system, as they are expensive and time consuming to build.

5.4.3 Knowledge Acquisition

Hayes-Roth *et al.* (1983) listed five stages in the production of a knowledge-based system; problem identification, conceptualisation, formalisation, implementation and testing (Figure 5.7). The first three stages of system building – identification, conceptualisation and formalisation – are part of knowledge acquisition.

Problem identification usually involves a single domain expert and a knowledge engineer. The domain expert tells the knowledge engineer about their domain and their knowledge, and the knowledge engineer restates what they have understood about the problem. Then the knowledge engineer must try to define the problem that is to be solved, and to try to characterise the supporting knowledge structures. The domain expert will go through some typical problems, verbalising their reasoning process, and the knowledge engineer tries to identify the key elements of the problem. However, description of case studies is often not enough, as most experts are unable to give clear, reliable accounts of how they solve a design problem (Hayes-Roth *et al.*, 1983). Watching experts, as well as listening, often leads to a greater understanding.

The concepts and relations identified in the identification stage are made explicit during conceptualisation. Then key concepts and goals need to be written down and appropriate strategies for different problems determined. At this stage the knowledge engineer should start to have some ideas about the representation of the knowledge, and the software tools that would be most appropriate.

However, it is not until the formalisation stage that the concepts, subproblems and information flow characteristics need to be represented in a form amenable to a software tool. At this stage understanding the nature of the data is important; is the data sparse or plentiful, is there uncertainty in the data, how is the data acquired, is the data reliable and complete? Knowledge is gathered from many sources; textbooks, research reports, *etc.*, as well as experts. By the end of the formalisation stage the structure of the knowledge-based system should be determined and the knowledge engineer should be ready to produce rules to embody the knowledge.

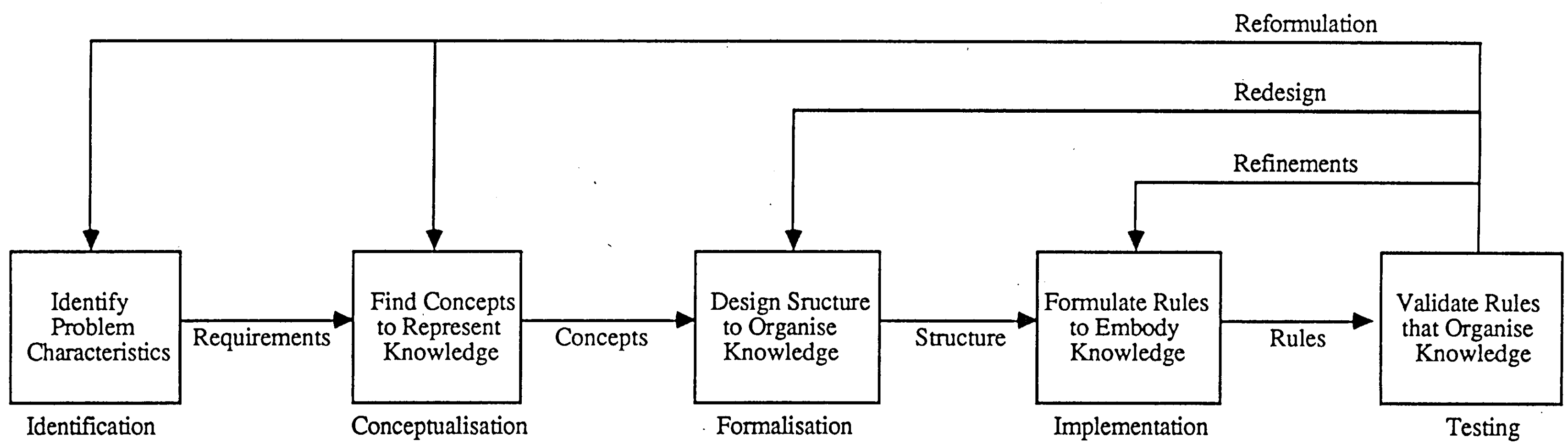


Figure 5.7 Stages of knowledge-based system development (after Hayes-Roth *et al.*, 1983).

5.4.4 System Implementation

Once knowledge has been elicited, the programming can begin using the chosen representation scheme. The most important thing with a representation scheme is that it ought to allow the most natural representation of the knowledge and have control strategies that most closely match those of experts. Efficiency of a program is not a prime constraint on the selection.

There are three general types of development environment. The first is a general purpose programming language, such as FORTRAN, PASCAL, C, LISP and PROLOG. To date LISP and PROLOG have been the favoured programming languages, although it is entirely possible to build a knowledge-based system using most programming languages.

The second approach is to use expert system *shells*. An expert system shell is a program that contains two of the three components of a knowledge-based system. The program has a working memory and an inference engine but lacks knowledge. The user develops knowledge bases using an editor, which is usually built into the system, in a language that is usually a subset of natural language. There are usually other development aids available also, such as debugging tools, user interface development facilities and explanation system tools, but all at a low level of sophistication. Originally, shells were developed for specialist LISP workstations and were very expensive and complicated to use. More recently, shells for micro-computers have become available, which are much simpler to use, yet retain many of the features of the more expensive programs. Table 5.1 gives a comparison between some of the more widely used shells available.

The advantage of using a shell is that the level of effort that must be applied to the development of support structures is greatly reduced, as they are built into the system. This allows the user to concentrate on gathering the domain knowledge and producing a complete program. However, as the size of the developed program increases, the restrictive nature of the representation schemes and inference mechanisms available may become excessive. At this point, if the general concept of the system has been proven, then it is best to move to a more complex development environment.

The third approach is to use a complex development environment, which can do all that a shell can do, but also contains additional facilities. A development

	Small System		Medium System		Large System	
Attribute	Expertease	Insight 2+	M1	Rulemaster	Rosie	S1
Approx. Cost	$695 (IBM-PC)	$495 (IBM-PC)	$5000 (IBM-PC)	$10,000 (IBM-PC)	$200 (VAX)	$50,000 (VAX)
Ability to handle maths	No	Yes (directly)	Yes (indirectly)	Yes (indirectly)	No	Yes (indirectly)
Ability to interface with other software	No	Yes (directly)	Yes (indirectly)	Yes (indirectly)	No	Yes (indirectly)
Explanation facility	None	Extensive	Limited	Extensive	Very limited	Extensive
Overall friendliness	Good friend	Good friend	A friend	A friend	Fair-weather friend	Very good friend
Documentation	Outstanding	Outstanding	Sufficient	Meagre	Sufficient	Sufficient
User support	Limited	Available	Available	Available	None	Available

Table 5.1 Knowledge-based system tool (shell) comparison (after Ludvigsen *et al.*, 1986).

environment usually allows multiple knowledge representation and control strategies, plus integrated editors, maintenance tools, debugging tools, user interface and explanation system development tools. These tools are expensive and restricted to specialist hardware like workstations, and can be seen as second generation versions of the original expert system shells. They also require trained users or knowledge engineers to operate them.

Once a prototype system works, it needs to be tested with a wide variety of examples to determine weaknesses in the knowledge base and inference structure. An experienced knowledge engineer will find out from the domain expert the type of problems that are likely to challenge the systems performance and reveal serious weaknesses or errors.

In the course of building a knowledge-based system there is almost constant revision, which may involve reformulation of concepts, redesign of the representation scheme or refinement of the final system. This refinement stage involves recycling through the implementation and testing stages. More drastic modifications may be needed, due to the fact that often it is not until a system is built that the knowledge engineer really understands the problem.

5.4.5 Conclusions

The development process outlined above is based on the philosophy as stated by Hayes-Roth *et al.* (1983). Fenves (1986a) argued that this will not be the predominant mode of knowledge-based system development in the future. Fenves (1986a) argued that explaining specialised domain knowledge to someone with no background in the domain is probably harder than developing a program oneself. With the quality of the expert system shells available today, it is likely that future knowledge-based systems will be developed by application programmers, educated in their respective application domains. The development process, however, will probably be similar to that outlined above with the program developers asking themselves the questions that a knowledge engineer would ask an expert previously.

5.5 Application Areas

5.5.1 Interpretation

Interpretation systems infer situation descriptions from various observations (Fytas *et al.*, 1988). Many engineering disciplines, including mining and geotechnical engineering, gather and interpret existing conditions as a first step in design. In geotechnical engineering much effort is devoted to the interpretation of field conditions measured by various instruments. In mining, data exists from diamond drill cores and consists not only of mineral grades but also of material properties of the rock such as the intact uniaxial compressive strength. In the following discussion of interpretation systems, the emphasis is placed on systems developed or proposed in the field of geotechnical engineering.

PROSPECTOR (Duda et al., *1979)*

PROSPECTOR is a computerised geological consultant system that is intended to aid trained geologists in evaluating sites for particular ore deposit types. PROSPECTOR was one of the earliest knowledge-based systems and was developed as a research tool. PROSPECTOR originally contained rules on only five ore deposit models; Kuroko-type massive sulphide, Mississippi-Valley-type carbonate lead/zinc, near-continental-margin porphyry copper, Komatiitic nickel sulphide, Western-States sandstone uranium. PROSPECTOR has subsequently been expanded to include more ore deposit types. An example of a dialogue with PROSPECTOR is given in Figure 5.8 with an accompanying commentary.

Knowledge in PROSPECTOR is encoded using semantic networks, which connect field evidence to geological hypotheses. PROSPECTOR has a plausible reasoning scheme based on Bayesian decision theory and exploits Bayes rule. There are three quantities involving an evidence assertion, E, and a hypothesis assertion, H. These are the prior odds $O(H)$ on the hypothesis, the posterior odds $O(H|E)$ on the hypothesis (given that E is observed to be present), and a measure of sufficiency, LS. Bayes rule can then be stated as:

$$O(H|E) = LS \times O(H)$$

All underlined statements are those entered by the user. When the user has finished entering data, they type DONE. Numbers in brackets are the certainty of the assertion.

1. <u>There are carbonates</u>
   (carbonates) (4) — [The computer acknowledges the first volunteered evidence - the certainty is taken as 4]

2. <u>There is dolomite</u>
   (dolomite) (4)

3. <u>There is galena</u>
   (galena) (4)

4. <u>There might be sphalerite</u>
   (sphalerite) (2)

5. <u>Done</u> — [The user is finished volunteering evidence and PROSPECTOR takes the initiative. The data matches the Mississippi-valley model best. The computer then starts systematic consideration of that model.

You told me about Carbonates
- Carbonates (4) — [The computer reminds the user of volunteered evidence]

6 - To what degree do you believe that:
(*Prospective orebody lies in Carbonate sediments)? <u>3</u>

.
.
.
.

12 - To what degree do you believe that:
(*Evidence of metamorphism)? <u>WHY</u> — [The user is interested in the reasoning behind the question]

* The evidence of high-temperature mineralisation is discouraging for the prospective orebody to be Mississippi-valley type. However, if the high temperatures were due to subsequent metamorphism, then this discouraging evidence should be discounted or ignored.

.
.
.
.

Do you want to continue? <u>NO</u>

[Prospector then offers a summary]

On a scale from -5 to +5
My certainty for Mississippi-valley type deposit is 0.8995

[0.8995 would be interpreted as mildly encouraging]

Figure 5.8 Sample of dialogue with the PROSPECTOR knowledge-based system (after Hart *et al.*, 1978).

There is a complimentary equation describing the case where E is known to be absent, that is when $\sim E$ is true.

$$O(H|\sim E) = LN \times O(H)$$

where LN is the necessity measure.

The rules in PROSPECTOR are of the IF...THEN type but associated with each hypothesis is a value for LS and LN. Figure 5.9 shows a portion of a PROSPECTOR model, for a porphyry copper deposit, that describes the regional environment. There is a top-to-bottom development in terms of successive levels of assertions.

In use, the geologist enters into a dialogue with the program, which asks questions about the existence of various kinds of evidence. Answers are given as numbers, ranging from +5 for definitely present, to −5 for definitely absent. Entering 0 indicates that you have no opinion either way. Further details on the Bayesian reasoning process can be found in Duda *et al.* (1979).

PROSPECTOR demonstrated that it was of practical use when it was able to recognise a hidden mineral deposit (Campbell *et al.*, 1982). Mount Tolman, in Washington state, U.S.A., was a known porphyry molybdenum deposit whose extent was not fully understood. PROSPECTOR utilised the geological, geophysical and geochemical information supplied by the group that had finished exploration in 1978. PROSPECTOR delineated a much larger reserve than had been previously recognised, which was later proved by subsequent drilling.

CONE (Mullarkey and Fenves, 1986)

CONE is a knowledge-based system for the interpretation of data from a cone penetrometer. CONE has two main objectives: to classify soils based on cone penetrometer data and to infer soil shear strengths from the same data. A cone penetrometer is a conically-tipped penetration device that is either electronically or hydraulically pushed into a soil profile. As the cone penetrates the soil, the tip resistance and the frictional resistance along the side are measured continuously with depth.

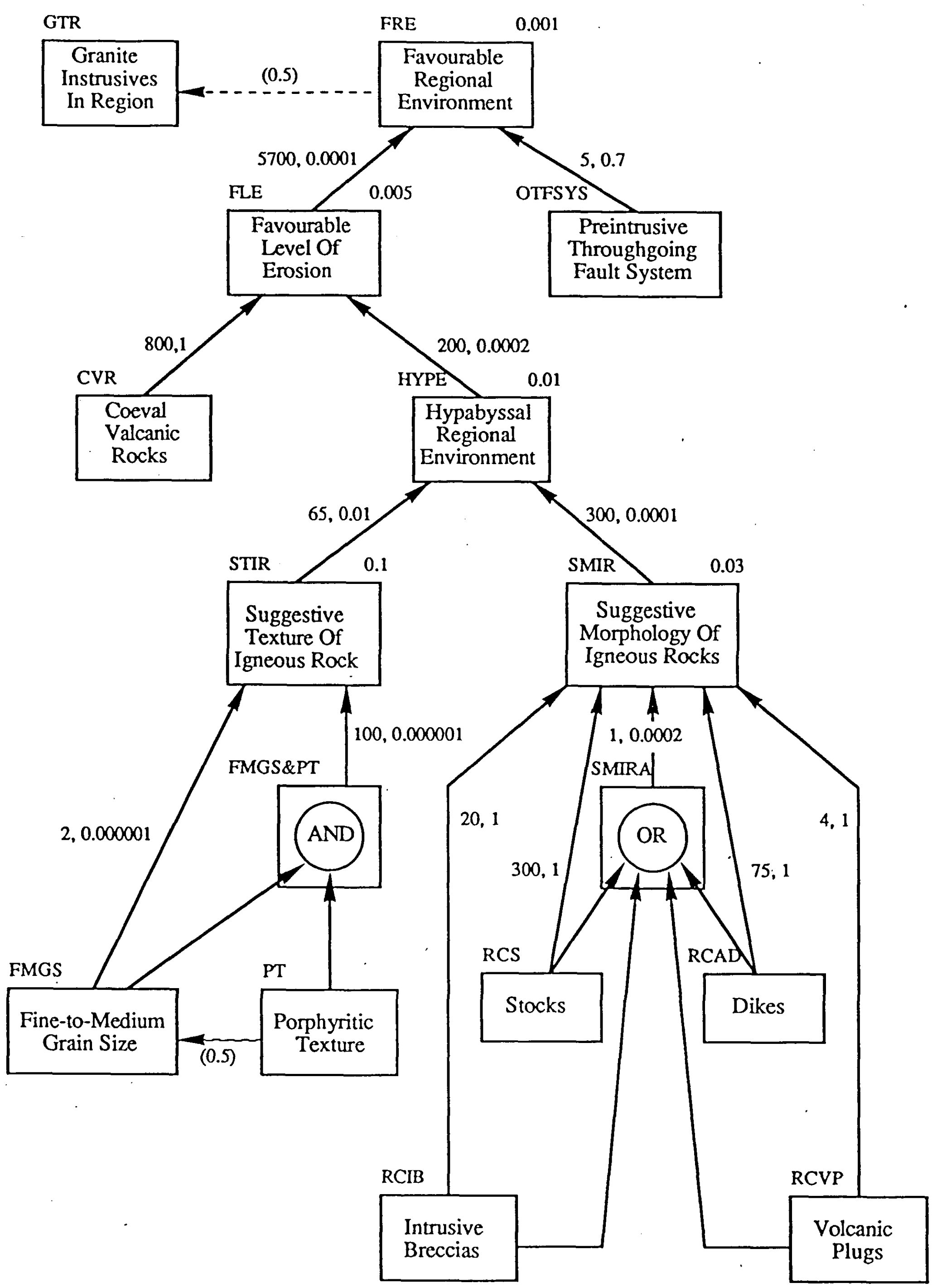


Figure 5.9 Portion of a PROSPECTOR model for porphyry copper deposits (after Duda *et al.*, 1979).

CONE begins by collecting preliminary information concerning the site and the raw input data. The raw data is then checked for validity and reasonableness. The system then classifies the soil and infers its shear strength. The system was developed using the OPS5 programming language, which utilises production rules, with both backward and forward chaining control mechanisms – backward chaining for the information gathering and forward chaining for the soil classification and shear strength estimation.

CONE uses *fuzzy* logic to combine the analytical results of multiple knowledge sources, to represent confidence, and to provide a common representation for natural language attributes. There are several ways of handling uncertainty; Bayesian probability theory (*e.g.* PROSPECTOR), certainty theory (*e.g.* MYCIN), and fuzzy logic. Fuzzy logic is a type of logic that uses graded or qualified statements rather than ones that are strictly true or false. The basic unit in fuzzy logic is the linguistic variable. An example of a linguistic variable, SOILTYPE, is given in Mullarkey and Fenves (1986). SOILTYPE could be represented as a three element fuzzy set with a characteristic function:

$$SOILTYPE = \{\mu_{sand}|Sand, \mu_{silt}|Silt, \mu_{clay}|Clay\}$$

Values of this linguistic variable would be of the form:

$$CLAY = \{0|Sand, 0|Silt, 1|Clay\}$$
$$SILTY - SAND = \{0.8|Sand, 0.4|Silt, 0|Clay\}$$

With these variables, fuzzy set theory allows union and intersection of sets.

The use of fuzzy logic is becoming increasingly popular in geotechnical and civil engineering (Mullarkey and Fenves, 1986). Nguyen (1983) described the use of fuzzy sets for a hazard index to mining excavations, and to evaluate RMR rock mass classification ratings, with the incorporation of expert knowledge. Yun and Huang (1987) described a fuzzy set algorithm for selecting the optimum mining method for a mine, based on the geotechnical properties of the rocks present.

Dipmeter Advisor (Smith and Baker, 1983)

Dipmeter Advisor is a commercial knowledge-based system for well-log interpretation. The dipmeter is a logging tool that measures the conductivity of rock in a number of directions around a borehole. Variations in conductivity can be correlated and combined with the measurements of the inclination and orientation of the tool, to estimate the dip and dip direction of various formation layers intersected by the the borehole. The system uses dipmeter patterns together with geological knowledge and data from other logs, emulating an expert in dipmeter interpretation.

Hyperspectral Image Analysis (Chiou, 1985)

NASA developed an image-based geological knowledge-based system for the hyperspectral analysis of remotely sensed satellite images. The purpose of the program was to identify directly from these images the surface mineral contents. The system relies on the fact that different mineral types absorb different parts of the spectral curve. The original system had ten radical groups for the minerals and rocks (*e.g.*, silicates, carbonates, intermediate igneous rocks).

5.5.2 Intelligent Modelling Tools

Another type of interpretation program are intelligent modelling tools. These can assist in generating the appropriate analytical model, in result interpretation, and in evaluating analytical results for appropriateness. With proper facilities, these tools can be integrated directly with analysis programs to become intelligent pre- and post-processors

The knowledge-based system for crown pillar design, described in Chapter 6, is an example of such a system. Intelligent modelling tools are becoming extremely popular in engineering, as there already exists a large base of knowledge (in the form of algorithms, subroutines and programs) that have been written in procedural languages such as FORTRAN and BASIC. To rewrite such programs in the same language as the knowledge-based system would involve considerable time and expense. By using knowledge-based system programming techniques to build intelligent pre- and post-processors the existing programs can be utilised for design.

General purpose finite element programs are now commonly used in engineering. They are characterised by their complexity and comprehensiveness of modelling capabilities. These programs take many years to master and it is unlikely that a practitioner will ever use all of the available options. Fenves (1986b) argued that the role of the finite element analyst has now changed, such that their primary functions are now:

- modelling, *i.e.*, translating a physical problem into an appropriate computational mechanics tool, and

- interpreting the results obtained from the mathematical model in terms of physical performance.

Knowledge-based systems offer a means of capturing such expertise and hence produce an intelligent pre-processor to a finite element program. Pre- and post-processors have already been built for large finite element programs and have aided design by making the preparation and input of data, and interpretation of the results, relatively simple. The major problem that still exists is to ensure that the analysis reflects the true physical problem, as inexpert users can waste valuable computing time, and produce very misleading output. Two types of knowledge-based system can help to overcome this problem of misusing programs.

1. Intelligent Front-End System

This type of system guides the user to the most appropriate facility, data entry method and solution method, and helps with the preparation of input data. SACON (Bennet *et al.*, 1978) was the first finite element knowledge-based system, and this dealt with the problem of method selection. NASCON (Corlett *et al.*, 1984) addressed the wider issues of intelligent front-ends.

Various methodologies have been employed in building these front-ends. Rivlin *et al.* (1980) wrote a rule-based, backward chaining front-end in FORTRAN, so that it could directly connect with their finite element program MARC. Kissil and Kamel (1986) used a forward chaining rule-based system, using breadth first searching in a frame-type structure. Allwood (1986) used an expert system shell, ExpertEase, to develop a program for error analysis in data input to a structural analysis program.

2. Specification and Modelling Aids

This type of knowledge-based system is aimed at the person who is reasonably expert in using the finite element program. The knowledge-based system identifies the important features of the real-world problem and guides the user in how to represent these in modelling terms. FEASA (Taig, 1986) was the first finite element knowledge-based system of this type. Taig (1986) used the expert system shell SAVOIR , which has a rule based structure. The system contained over 2000 rules and performed the following functions:

- elicits a description of the problem, its loading conditions, material properties and geometry,
- recommends representation methods for structures,
- advises and records decisions on finite element modelling of the structure, and
- negotiates, interactively, the problem size and mesh size, in order to keep the analysis within resource limitations.

These front-ends and modelling aids are closely related to interpretation and diagnosis systems (see Section 5.5.3), in that they are generally data-driven, and their task is to select the appropriate analysis model and to correctly interpret the analysis results. These systems are not planning or design systems because they classify a given modelling problem from a class of known models that are represented in its knowledge base, rather than having general design knowledge. More advanced concepts using frame-based structures and a blackboard architecture have been described by Fenves (1986b). Such a system would contain knowledge from many sources. There would be declarative knowledge, representing either empirical or heuristic knowledge, and procedural knowledge, representing those components of the domain knowledge where the problem solving strategy is well established.

Emphasis here has been placed on intelligent front-ends to stress analysis programs, and in particular to finite element programs. There are front-ends to other types of engineering analysis programs but a complete description of these is beyond the scope of this thesis. In the field of geotechnical engineering there are many such programs, *e.g.*, RETWALL. RETWALL (Siller, 1987) is a system for choosing the appropriate retaining wall for a particular structure. The wall that is appropriate depends on several factors, including wall height, soil conditions, and the location of the wall. The system also has the ability of performing the actual design calculations.

5.5.3 Diagnosis

Diagnosis systems infer system malfunctions from various observations. In engineering there are many applications for such knowledge-based systems, with equipment troubleshooting being one of the most obvious. This is especially true for mining engineering, in that mines are often remote and maintenance of large, complicated equipment can be expensive. It is also true that there are few experts available in the field for such work. To date no commercial systems of this kind have been developed for mining equipment, although General Electric developed the DELTA system to help railroad maintenance (Harmon and King, 1985).

Other types of diagnostic programs, however, have been developed in many fields. One of the main areas for diagnostic programs has been in medicine, where the program MYCIN (Shortliffe, 1976) was one of the earliest and most famous knowledge-based systems to have been built. MYCIN is a consultation system designed to diagnose bacterial infections and recommend antibiotic treatment. The program is a rule-based system, that links the patient data to the infection hypotheses (Figure 5.2 shows a typical rule from MYCIN). The system is modular in nature which allowed it to grow incrementally over a period of time.

Drilling Advisor (Courteille *et al.*, 1986) is a knowledge-based system for the diagnosis and possible treatment of problems encountered during the drilling of oil wells. The system begins by gathering preliminary data in a consultation, asking for information such as the nature of the formation, description of the situation, and a list of symptoms. A preliminary diagnosis excludes the least probable causes and orders the rest into a suspected-cause list. The system then asks more detailed questions, evaluating and eliminating options as it goes along, to provide a final list of possible causes, along with the appropriate treatment to be undertaken.

Scheck *et al.* (1987) developed a surface mine blast design and consultant system. The system is comprised of two modules: one to design a blast given the geological data, whilst the other module analyses blast vibration problems and recommends methods to solve the problems. At present this system is undergoing field trials.

5.5.4 Planning and Design

In engineering, the problem to be addressed is often concerned with planning and design, *i.e.*, problems that are normally thought to require human intelligence to solve (Duda and Shortliffe, 1983). Gero and Coyne (1986) developed one of the earliest design synthesis systems, for the design of kitchen layouts. In one mode, the knowledge-based system checked a designers layout for compliance with rules of good practice. In another mode the system checked the design as it progressed and completed any detail. In a third mode, the system made decisions on the basis of a dialogue with the designer, and employed its rules of good practice to produce prototypical solutions.

The field of civil engineering has been active in developing many planning and design knowledge-based systems. Rouhani and Kangari (1987) produced a landfill site selection program which acted as an assistant to a waste disposal manager in the preliminary stages of site selection. The system took many conflicting factors into consideration and evaluated the advantages and disadvantages of various sites, using rules in its knowledge base, to determine the optimum site. Structural design programs, such as those developed by Burgoyne and Shan (1987), are based on the availability of guidelines in civil engineering about design practice. Burgoyne and Shan (1987) developed a prestressed concrete bridge knowledge-based system, with rules governing the selection of different types of bridge deck, choice of prestressing methods, cross section shapes and construction methods. These guidelines are often written in large manuals and are difficult to understand and know fully, even for experienced engineers. These systems codify this knowledge in a form that does not diminish the role of the engineer but, and are intended merely to act as intelligent assistants to the engineer.

Another feature of planning and design, is that traditional rule-based representation schemes are often inadequate and that frame-based representation is often preferred. MacRandal (1987) developed an intelligent front-end to a large and complex building appraisal package in the field of dynamic energy modelling. The system transformed the users ideas into a form that was usable by the design package. A blackboard approach was taken (Nii, 1986) as a means of communicating between independent modules (Figure 5.10 shows the system architecture). The advantage of this approach is that once the blackboard is specified, the independent modules can be developed separately.

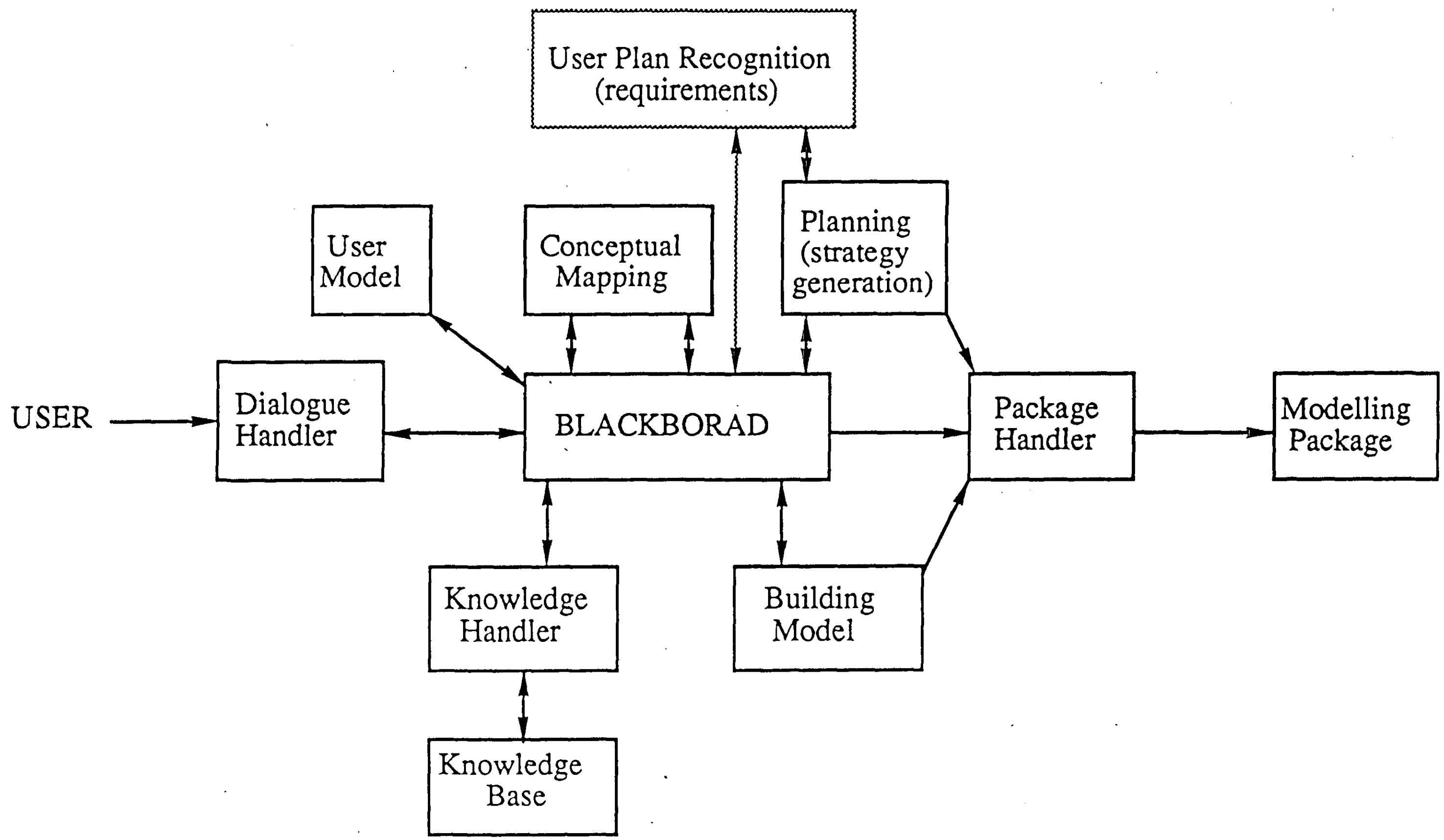


Figure 5.10 Outline of the system architecture of a complex knowledge-based system for design, utilising frame-based representation and a blackboard approach (after MacRandal, 1987).

In the field of mining engineering, few planning programs have been reported, exceptions being CHOOZ (Britton, 1987) and a decision support system developed by Nenonen *et al.* (1988). CHOOZ automatically searches through a central database of mine manpower characteristics, to produce work crews, premium work lists, and can also suggest strategies for deploying the workforce for maximum productive effort. The user simply has to enter the boundary variables (*e.g.,* jobs to be filled, skill level needed), plus any workers who have already been assigned to jobs. Nenonen *et al.* (1988) described a knowledge-based system for surface mine planning. The system consists of existing conventional software (*e.g.*, simulation software) for which intelligent front-ends were developed. Allied to these were other modules for equipment selection (based on geotechnical properties of the rock to be excavated) and for evaluating the performance of various load/haul combinations. The system was intended to be used as an aid to mine planners, who would try their ideas out, whilst receiving intelligent advice online.

Ghosh *et al.* (1987) developed a knowledge-based system for the design of rock bolt layouts in underground coal mines. The system enters into a dialogue with the user and produces a suggested bolting layout. The system was tested against various case studies in the literature and appeared to agree quite closely. Sinha and Sengupta (1989) developed a slope stability knowledge-based system. The program acts in a similar way to that of Ghosh *et al.* (1987) in that preliminary geotechnical data is gathered in order to try to identify failure mechanisms. The slope stability program can then give advice about remedial action that could be taken, based on the mechanics of each type of landslide. The program is envisaged to be further expanded by linking it to numerical models for quantitative slope stability calculations.

5.6 Conclusions

Developing knowledge-based systems is not a simple task, and can be both expensive and time consuming. In many situations, especially in planning and design work, knowledge is difficult to elicit, as few methodologies have been written. However, there are benefits to be gained from attempting to develop knowledge-based systems. The systems can perform routine tasks with a high reliability and consistency and do not tire. The process of eliciting the knowledge leads to a deepening of understanding in the subject by the practitioner, and provides a means of sharing and propagating knowledge within an organisation, even after a practitioner has left. The main benefit to the user is that it frees them from routine, time-consuming tasks, and allows them to concentrate on more creative tasks, or to consider the tasks that occur rarely, in more detail. Even if a successful knowledge-based system is not developed, an organisation will probably have gained benefit, because facts, rules, methodologies that have not been explicitly stated before, will now have been.

This chapter has described many of the applications for which knowledge-based systems have been used. There are potentially many areas where knowledge-based systems could be of use in engineering rock mechanics (Dershowitz and Einstein, 1984; Ramani and Prasad, 1987), but this thesis concentrates on one area in particular, that of intelligent tools for numerical modelling. Knowledge-based system techniques afford a means of collecting the available expertise in rock mechanics numerical modelling, and encoding it into a computer program. The knowledge-based system that is described in Chapter 6 draws from all aspects of knowledge-based system technology, but is essentially an intelligent modelling aid and front-end to a suite of boundary element programs.

CHAPTER 6
DEVELOPMENT OF A KNOWLEDGE-BASED SYSTEM FOR CROWN PILLAR DESIGN

6.1 Introduction

Numerical modelling has become an integral part of engineering rock mechanics. There exists thousands of numerical programs (Plischke, 1988), all with a variety of modes of use, and all with their own particular assumptions and limitations. To date, these programs have not been used routinely in mine design, but they are increasingly being incorporated into mine design methodologies. There are many reasons why these programs have not been more widely used, but the main problems have been that there were no formalised design methodologies, and that the numerical programs available were only operable by highly skilled users, and commonly only by the program writer.

The research described in this thesis has attempted to address these problems within the context of open stope mine design. Chapter 2 described the range and applicability of the different numerical methods available, and concluded that the boundary element method was the most suitable method for open stope and pillar design. A suite of boundary element programs were selected and tested, to assess their suitability for open stope design problems, concentrating specifically on design in narrow, steeply dipping, tabular orebodies. Chapters 3 and 4 critically reviewed the existing design methodologies for open stope and pillar design, respectively. In each of these chapters, a new advanced design methodology was developed that incorporated the use of simple boundary element programs. These methodologies were specifically developed for open stopes and pillars in narrow, steeply dipping, tabular orebodies, but are to some degree applicable to any open stope.

For these design methodologies to be of use to mining engineers, the problem of using the boundary element programs (the fact that they require expertise to use) has to be considered. The task of modelling an open stope and pillar layout for a mine requires judgement by a modeller, based on accumulated experience, that is largely heuristic in nature. In Chapter 5, the new technology of knowledge-based systems was discussed and was shown to provide a framework, whereby expert modelling heuristics and compiled domain-specific knowledge could be implemented in a computer program, that could then be combined with numerical analysis programs.

The problem that is to be addressed, using a knowledge-based systems approach, is first defined, and then conceptualised in Section 6.2. There follows a discussion about the selection of the most appropriate development system. In Section 6.3 the knowledge and logic, contained in the developed crown pillar design knowledge-based system, is described. Tentative conceptual models, for the behaviour of rock around open stopes, are developed, to which are linked appropriate numerical analysis techniques. Attention is then focused on the design of crown pillars in narrow, steeply dipping, tabular orebodies. The boundary element programs, that were identified in Section 2.3.5 as being of most use in crown pillar design, are further investigated. The specific circumstances under which each program should be used are determined. Intelligent front-ends to two two-dimensional boundary element programs are developed, that enable the datafile for a numerical analysis to be written by the system itself.

Finally, in Section 6.4 conclusions about the developed knowledge-based system are given. The system is viewed as being only a prototype, designed to prove the potential and use of knowledge-based system technology, for stope design methodologies that incorporate numerical analysis. Some ideas are given about possible future developments of the system, as well as to the possibility of incorporating such a system into a knowledge-based system for underground mine design.

6.2 Problem Definition

6.2.1 Problem Identification

It is important to evaluate fully the suitability of a problem for solution using a knowledge-based systems approach. The aim should not to be to convert algorithmically solvable problems into a form that could be solved using knowledge-based system techniques. Rather, the algorithmic part of a problem should be separated out and only those areas that require large amounts of knowledge and expertise should be part of a knowledge-based system. Boundary element modelling of open stope layouts meets all the criteria usually presented for a good knowledge-based system application. These were defined by Fenves (1986b) to be:

- Algorithmic solution patently inapplicable or would be too cumbersome or restrictive,
- There are recognisable experts, with notable better performance than novices,
- Experts can be debriefed, or major elements of expertise are available from publications,
- The basic components of knowledge are known, but the task domain is not formalised, and
- There is great potential utility.

It is also important to select a narrow specialised domain, often that is a subset of a much larger problem. Developing a knowledge-based system for all aspects of open stope design would be a large scale task and would take many years. Problems also exist if such a system were to incorporate elements of spatial reasoning, whereby a program can reason about points in space and their relation to other points. Spatial reasoning is used in robotics but is beginning to find use in other application areas. Akiner (1985) attempted to develop a knowledge-based computer aided design (CAD) system that incorporated object and space modelling and Buisson (1987) developed a knowledge-based system for avalanche analysis that incorporated elements of spatial reasoning. However, research into incorporating spatial reasoning into engineering knowledge-based systems is still at a very early stage.

For these reasons it was decided to demonstrate the applicability of knowledge-based system techniques in open stope design by selecting a narrow domain, namely crown pillar design in narrow, steeply dipping, tabular orebodies. The concepts contained within this system, together with the modularity of its design, ensure that the system could be enlarged subsequently, by increasing the knowledge in the knowledge base to cover crown pillar design in all orebody types, as well as other aspects of open stope layout, such as rib pillar design, hangingwall design, stope dimensioning, stope sequencing, *etc.*

6.2.2 Problem Conceptualisation

The main problem mining engineers would encounter when using the developed open stope and pillar design methodologies, described in Sections 3.5 and 4.4, respectively, was identified to be in the use of the numerical programs. More specifically, mine engineers, who were non-expert numerical modellers, would encounter difficulties in:

- determining the type of analysis required,
- producing a model of the rock mass,
- preparing input data for the numerical analysis, and
- interpreting the output from the program in terms of rock mass response.

Earlier work in the development of knowledge-based systems for finite element programs addressed one of the above problems. Bennet *et al.* (1978), Rivlin *et al.* (1980) and Corlett *et al.* (1984) all developed *intelligent front-end systems* (Section 5.5.2). These intelligent front-end systems were knowledge-based systems designed for someone who knew how to represent a physical problem in the appropriate finite element terms, but who was not an expert user of the particular finite element package. The systems guided the user to the appropriate modelling facilities (*e.g.*, the correct element type to use) and helped in writing the data file for the programs. Taig (1986) developed a *specification and modelling aid* (Section 5.5.2) that was designed for someone who was not an expert modeller. The system helped the user to identify important features of a real world problem and gave advice in recasting the problem in terms that a finite element program could understand.

Mine planning engineers are commonly neither expert modellers nor expert users of numerical analysis programs. Thus, a developed knowledge-based system for crown pillar design would have to contain both an intelligent front-end to a suite of boundary element programs, as well as a modelling aid for engineers. The intelligent front-end to the boundary element programs would consist of knowledge from manuals as well as judgmental knowledge, gained from personal use of the programs. The modelling aid would be given the material properties of a rock mass, together with a stope layout design, and using these would determine the correct type of analysis to use. This assessment would be based not only on the rock mass conditions, but also on the geometry of the stopes. This knowledge is rarely mentioned in program manuals and is only sketchily treated in the literature. This modelling knowledge was obtained from experts in rock mechanics modelling at Imperial College, London. In order to model a rock mass, conceptual modelling (Section 3.3.3) was chosen as the best approach to encapsulate the important features of the rock mass. No conceptual models for rock around open stopes existed, so a series of models had to be developed. These required basic information about stope size and rock mass conditions. Modelling should then take place within the context of the conceptual model. For each model, the appropriate numerical analysis techniques had to be determined, together with the defining of exceptions and limitations, and how these should be treated.

Thus, the program that was to be developed was aimed at both the pre-analysis and analysis stage of numerical modelling of crown pillars. By combining a modelling aid, with intelligent front-ends to a suite of boundary element programs, mining engineers, who were non-expert numerical modellers, should then be able to perform analyses that were numerically correct, as well as truly reflecting the rock mass conditions present, and should then be able to use the results to enhance the mine design. The knowledge-based system that was developed was designed to prevent unrealistic analyses, and hence prevent potentially misleading solutions.

6.3 The Development of a Knowledge-Based System for Crown Pillar Design

6.3.1 Development System Selection

As was discussed in Section 5.4.4 there are several possible approaches to the development of a knowledge-based system. The system can be developed using a general purpose programming language (*e.g.* FORTRAN, PASCAL, LISP, PROLOG), using a simple expert system shell or using a complex development environment program. The aim of the research was to develop methodologies for open stope and pillar design, concentrating specifically on design for narrow, steeply dipping, tabular orebodies. The developed methodologies utilised numerical modelling as an integral part of the design, but for these methodologies to be used effectively in practice a means had to be found whereby the inherent difficulty in using the numerical programs could be overcome.

The form of the problem, and the availability of a large source of knowledge on numerical modelling, suggested that it might be possible to develop a knowledge-based system. In order to investigate the possibility of building a system, a tool was required that would allow rapid prototyping of programs, and which would involve the least amount of programming. Developing a system using a general purpose programming language allows the system builder to handcraft a program to meet their particular needs, but it requires that all the elements of a knowledge-based system – knowledge base, inference engine and working memory (Section 5.2) – have to be programmed. The advantage of an expert system shell is that it already contains the knowledge base structure and reasoning strategy and requires only the knowledge to be inputted. A complex development environment program was ruled out as being both difficult and time consuming to learn how to use properly, as well as being prohibitively expensive. Hence, it was decided to use a simple expert system shell to develop the knowledge-based system.

There are literally hundreds of shells available and they vary in size, complexity and price. The aim when developing a knowledge-based system with a shell is to define the criteria that a shell should possess for the particular problem. Different shells have different reasoning strategies – some only have forward chaining, some backward chaining (Section 5.2.3), whilst others allow a mixture. For engineering purposes, the reasoning strategy should not be restricted to one type, as this tends to force the programmer into restructuring the problem in a manner that fits the shell. Shells that allow a mixture of reasoning strategies are more flexible, and reflect more closely the nature of a problem.

A shell should provide facilities that make the programming easier, such as built-in editor, debugger and transparency in the rule structure, such that the logic of the rules can be checked. The ability to use probabilities to express uncertainty in answers to questions is useful in many applications, where answers are not always true or false. This is especially true in rock mechanics, where there is often great uncertainty in data, due to the difficulty and expense in collecting large amounts of data, and the problem of how representative the data actually are.

In engineering, the ability to handle complex mathematics as part of the rules is a useful feature, as it removes the need to write separate algorithms that are external to the knowledge-based system. The main criterion when choosing a shell is whether the shell allows information to be incorporated in the system in a manner that is as close as possible to the way an expert would express themselves when training an assistant, *i.e.*, in as natural a way as possible.

The shell INSIGHT 2+ (later renamed LEVEL5) was selected as the most appropriate tool. It met all the criteria stated above, as well as having good direct interfaces to databases (dBase 2 and dBase 3 files) and procedural programs (PASCAL, FORTRAN). Further description of the shell, which is available for IBM Personal Computers, Apple Macintoshs and DEC VAX machines, is given in Adeli (1987). The ability to access external procedural programs directly was of particular use, as the boundary element programs were all written in FORTRAN. This is a common situation in engineering, where large numerical analysis programs written in FORTRAN or other high level languages exist. The cost of rewriting these programs so that they can be an integral part of a knowledge-based system, written in a language like PROLOG, is prohibitive and counter-productive. Developing intelligent front-ends to numerical programs requires no rewriting, and allows knowledge-based systems to be directly linked to these external programs. The knowledge base and algorithms are kept totally separate, which makes the understanding of the logic contained in the knowledge-based system much easier.

6.3.2 Conceptual Models for Crown Pillar Design

As was identified in Section 6.2.2, numerical modelling of a rock mass presents problems for non-expert modellers. In order to ensure that the essential features of a rock mass were taken into account in the modelling, and that the appropriate numerical analysis program was selected, it was necessary to develop a set of conceptual models of rock masses, for use in open stope design in hard rock mines.

The behaviour of a rock mass surrounding an excavation can best be determined by comparing the size and shape of the rock blocks formed by persistent joint sets, relative to the size of the excavation. Barton *et al.* (1974) proposed that the effect of block size could be quantified by the ratio RQD/J_n.

RQD is commonly assessed from drillcores. However, underground discontinuity mapping in all three dimensions, as described in Brady and Brown (1985), is recommended in order to obtain a fuller description of the orientation and spacing of all discontinuity sets. The RQD can then be determined using the correlation between RQD and the volumetric joint count, J_v, of the form:

$$RQD = 115 - 3.3J_v \tag{6.1}$$

where $RQD = 100$, for $J_v < 4.5$, and $RQD = 0$ for $J_v > 33$

Palmstrom (1985) defined the volumetric joint count, J_v, as:

$$J_v = \frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{S_3} + \ldots + \frac{1}{S_n} \tag{6.2}$$

where S_1, S_2, S_3 are the spacing of joint sets 1, 2, 3 *etc.* Palmstrom (1985) recommended that where foliation, slatey cleavage or bedding occurred, then these joints should be considered not as a joint set but as random fractures, except where these joints were strongly developed, where they should then be counted as a complete joint set. The parameter J_n is part of the NGI classification system (Section 3.4.1) and is related to the number of discontinuity sets present in the rock mass.

The hydraulic radius (Section 3.4.1), which was defined as the ratio of the surface area to the perimeter of a plane, is a means of taking into account the effect of both the size and shape of a stope surface. As the length to width ratio of a plane increases beyond 4 : 1 the hydraulic radius remains relatively constant. This corresponds to the best shape (long and narrow) for a stope plane of given area, from a stability point of view.

Potvin (1988a) studied the failure mechanisms around a series of open stopes and for each stope determined the ratio of the block size parameter, RQD/J_n, to the hydraulic radius of each open stope surface being considered. Potvin (1988a) was able to show that different failure mechanisms had quite different ratios of RQD/J_n to hydraulic radius, and so could be used as a guide to determining likely failure mechanisms around open stopes.

Using this ratio, three basic conceptual models of rock mass behaviour around open stopes have been developed – intact rock mass, jointed rock mass, and discrete rock mass behaviour. In the intact rock mass model, the rock was assumed to exhibit continuum behaviour, and could be modelled using linear-elastic analysis. In the jointed rock mass model, the rock was assumed to exhibit pseudo-continuum behaviour (Section 1.2.3), and could be approximately modelled using linear-elastic analysis. In the discrete rock mass model, the rock mass was assumed to exhibit discontinuum behaviour, and should be modelled as such, *e.g.*, using the distinct element method (Section 2.4).

Linked to each conceptual model is an appropriate numerical method. For the discrete model, the distinct element method is recommended, but in the actual developed system, boundary element analysis can be used in such a situation if the user wishes to. This is because a distinct element program for use on a micro-computer was not available. In the next section, the selection of the most appropriate analysis method for the intact and jointed conceptual models is discussed.

6.3.3 Crown Pillar Numerical Analysis Selection

Given that the crown pillar is amenable to continuum or pseudo-continuum analysis, then the actual numerical programs have to be determined. As was described in Section 2.3.5, six two-dimensional and a three dimensional boundary element program have been adapted for use on an IBM personal computer. These programs were selected for applicability to narrow, steeply dipping, tabular orebodies and were able to address most aspects of numerical analysis of open stopes. In the description of each program in Section 2.3.5 their suitability for use in pillar design was discussed, together with their individual numerical features. It was concluded that the program TWOFS was suitable for conventional two-dimensional plane strain analysis, BEM11 for complete plane strain analysis, and MINTAB for three-dimensional analysis.

The dimensions of the stopes above and below the crown pillar are used to determine whether plane strain conditions exist in the crown pillar. If plane strain conditions do exist (as occurs in the central section of long, thin pillars) then two-dimensional analysis is possible. If plane strain conditions do not exist, *e.g.*, in an orebody where small isolated pillars are used for support, a three-dimensional program is recommended. For narrow, tabular stopes MINTAB is recommended, but because it is a displacement discontinuity program it cannot give the stress distribution within the pillar, as the pillar is assumed to have zero thickness in the method. The methodology given in Section 3.5 described how MINTAB can be used to optimise pillar size and location. For open stopes that are more cubic in shape a true three-dimensional program, such as the one described by Watson and Cowling (1985), is recommended. This program is not part of the suite of boundary element programs, as it is a large complex program that cannot be run on a micro-computer.

Where plane strain conditions exist and two-dimensional analysis is possible then there are two possible programs to use – TWOFS and BEM11. Brady (1979) showed that when the long axis of an orebody was inclined at an angle greater than 20° to an *in situ* stress then complete plane strain analysis, using a program such as BEM11 was necessary. Where the long axis of an orebody was inclined at an angle less than 20° to an *in situ* stress then a program such as TWOFS could be used. The reason that the more general program, BEM11, is not always used is that it is considerably slower than TWOFS. A flowchart of the modelling aid for crown pillar analysis, containing the conceptual model selection and numerical analysis selection is given in Figure 6.1.

6.3.4 Intelligent Front-Ends to Numerical Analysis Programs

Two similar intelligent front-ends to two-dimensional boundary element programs were developed, one for TWOFS and the other for BEM11. These front-ends were designed to act as intelligent user manuals, that would enter into a dialogue with the user and obtain the necessary information to conduct a numerical analysis. The intelligent front-ends would then write the analysis datafile, ensuring that no errors in the coding of the data had been made. However, the systems could not eliminate all errors, as they are reliant on the data that are given to them, but the systems do contain internal checks and the inputted values are displayed. This allows the user the opportunity to check and possibly correct the input data. The front-ends to both TWOFS and BEM11 are similar to each other in their approach to knowledge elicitation, and for this reason, only the intelligent front-end to TWOFS is discussed.

The front-end first attempts to determine whether the design problem possesses any geometrical symmetry with respect to the chosen co-ordinate system, as TWOFS has the ability to take symmetry into account and so model only a half or quarter of the problem, depending on whether there are one or two planes

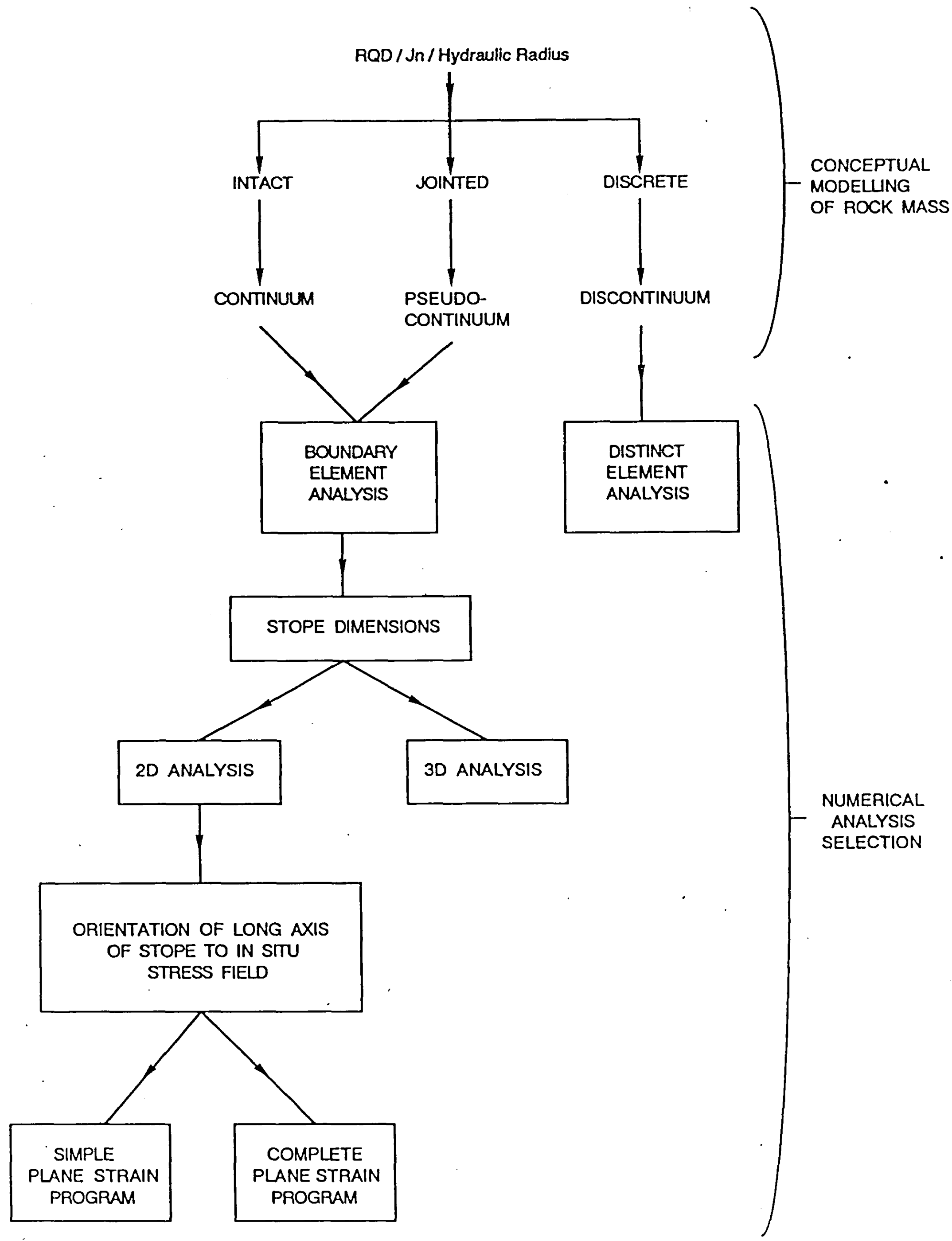


Figure 6.1 Flowchart of the developed modelling aid for crown pillar design. The rock mass is first conceptualised and then the appropriate numerical analysis program is selected.

of symmetry parallel to the x and/or y axes, respectively. For most open stope applications there will be no symmetry, as stopes are rarely symmetrical (Figure 6.2).

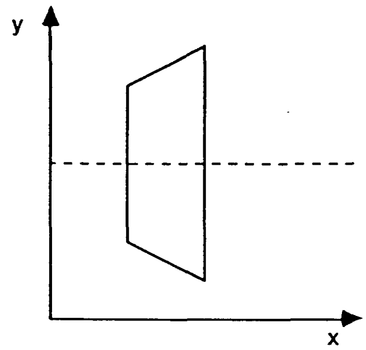
The front-end then attempts to define a failure criterion for the crown pillar. The front-end requests the pillar rock properties – uniaxial compressive strength, Young's modulus and Poisson's ratio. The front-end then tries to determine the rock mass rating (RMR') for the crown pillar (Section 3.5 – *Rock Mass Properties*, in order to determine the m and s constants for the Hoek-Brown failure criterion. Thus, a failure criterion for the crown pillar can be developed, which when used with the stresses determined from the numerical analysis, can be used to predict the stability of the crown pillar.

The front-end then asks the user to discretise the boundaries of the stopes above and below the crown pillar. For this, the user requires a vertical cross-section of the orebody, together with the co-ordinates of the corners of the stopes (Figure 6.3 shows the data that are required to discretise a hypothetical stope). The boundaries of each stope are divided into segments and along each segment a number of boundary elements are defined. Due to the limitations on the size of program that can be run using the disk operating system (DOS) of an IBM personal computer, only 100 elements are allowed in any one problem. Thus, the user has to decide in advance the best way to discretise the stope boundaries. Explanations are available online to the user describing the best approaches to boundary discretisation, and on the density of elements required. The user enters the number of elements along a segment and the start and end co-ordinates of the segment. Again there are explanations available online, about how a segment is divided into elements, and the co-ordinate system employed in the program.

Once the user has entered the boundary discretisation data, the user states where stresses and displacements are to be calculated. The stress and displacements are calculated at the intersection points of a grid that the user defines. Stresses and displacements can also be determined along extra internal lines if desired (Figure 6.4 shows the data that are required to delineate a grid and internal lines).

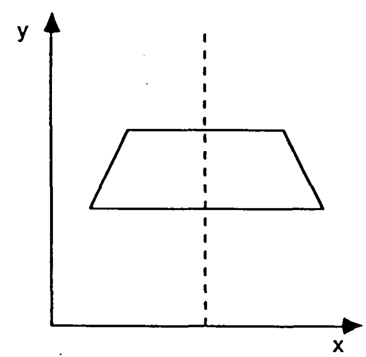
For crown pillar design the main area of interest is the area in the immediate vicinity of the crown pillar. However, sometimes data is required throughout the whole rock mass and the grid can be extended to cover a larger region. The use

(a)



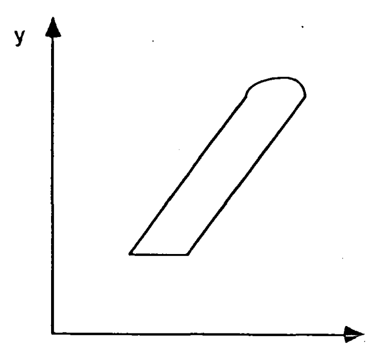
Stope has a line of symmetry parallel to x axis

(b)



Stope has line of symmetry parallel to y axis

(c)



Stope has no line of symmetry parallel to x or y axis.

Figure 6.2 Definition of the symmetry condition in the developed intelligent front-ends.

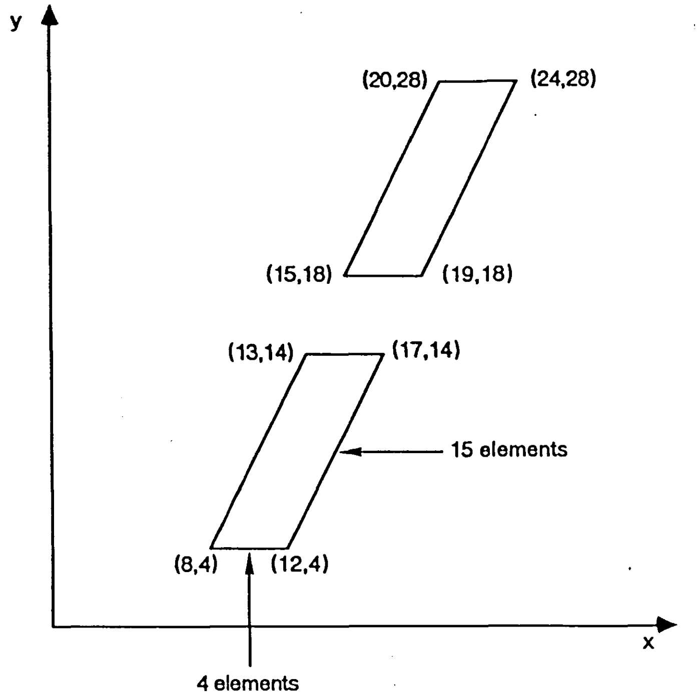


Figure 6.3 Example of the data required to discretise the stope boundaries in the developed intelligent front-ends.

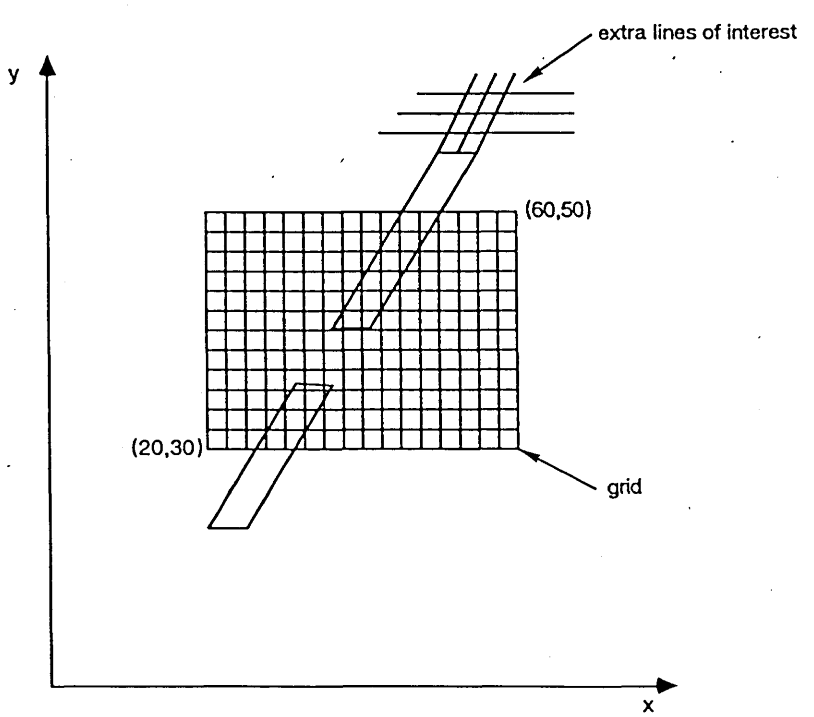


Figure 6.4 Example of the data required to delineate the grid and internal lines, where stresses and displacements are to be calculated, in the developed intelligent front-ends.

of extra internal points gives finer detail in areas where this is required. One example of this is in the centre of the pillar, where the average factor of safety against failure, as determined using the calculated stress distribution and the given failure criterion, can be used to predict the overall stability of the pillar (Section 4.4.2).

Finally, the front-end presents a short summary of the input data, giving the approximate length of time that the numerical analysis will take, and writes the input data to a datafile. The user can then run the numerical analysis using the datafile that they have just created. Alternatively, they can save the datafile for use at a later date. Details on running the numerical analysis program and saving the data file are given in online explanations. Unfortunately, due to the size of the numerical programs and the limitations of DOS, the user has to leave the INSIGHT 2+ environment in order to run the numerical analysis. A flowchart of the developed intelligent front-end is given in Figure 6.5.

6.3.5 The Design of the Developed Knowledge-based System

As was described in Section 6.3.3, the first part of the developed knowledge-based system for crown pillar design consists of a modelling aid for the numerical analysis of crown pillars. This modelling aid used the conceptual models that were developed in Section 6.3.2, together with compiled knowledge and heuristics about the selected boundary element programs. Figure 6.1 gives a flowchart of the modelling aid structure. The second part of the developed knowledge-based system consists of intelligent front-ends to the selected boundary element programs. The logic and structure of these front-ends were described in detail in Section 6.3.4 and schematically shown in Figure 6.5.

The user of the knowledge-based system is assumed to be a mining engineer, who is conversant with the terminology employed at metal mines. The user is assumed to wish to design a crown pillar in a narrow, steeply dipping tabular orebody. The user also requires a two-dimensional vertical cross-section and a longitudinal section, together with the co-ordinates of the orebody in space. Figure 6.6 is an example of the basic geometric information that a user must possess to use the system.

The knowledge-based system begins by entering into a dialogue with the user, in which initial geotechnical and geometrical data of the problem being considered,

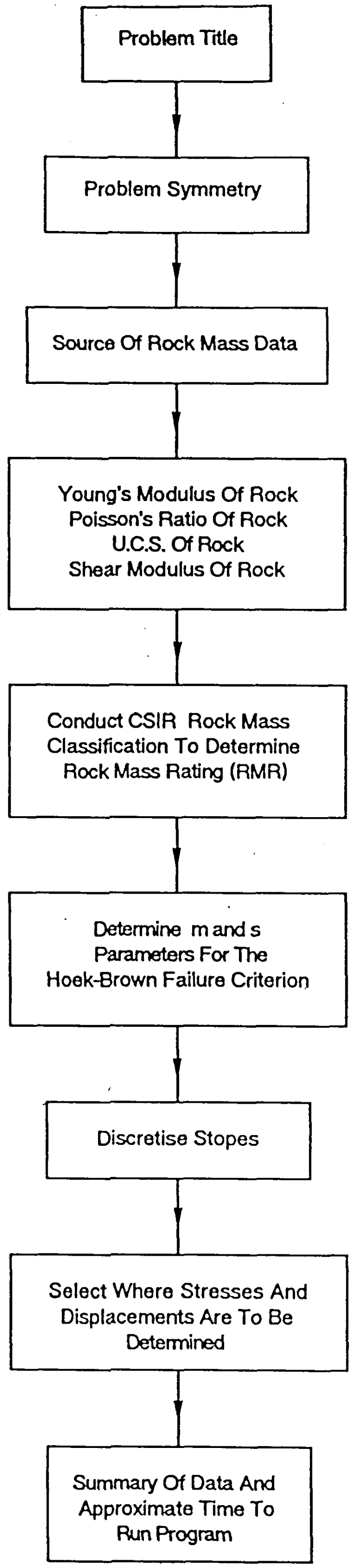


Figure 6.5 Flowchart of the intelligent front-end to the two-dimensional boundary element program TWOFS. The front-end to BEM11 has a similar structure.

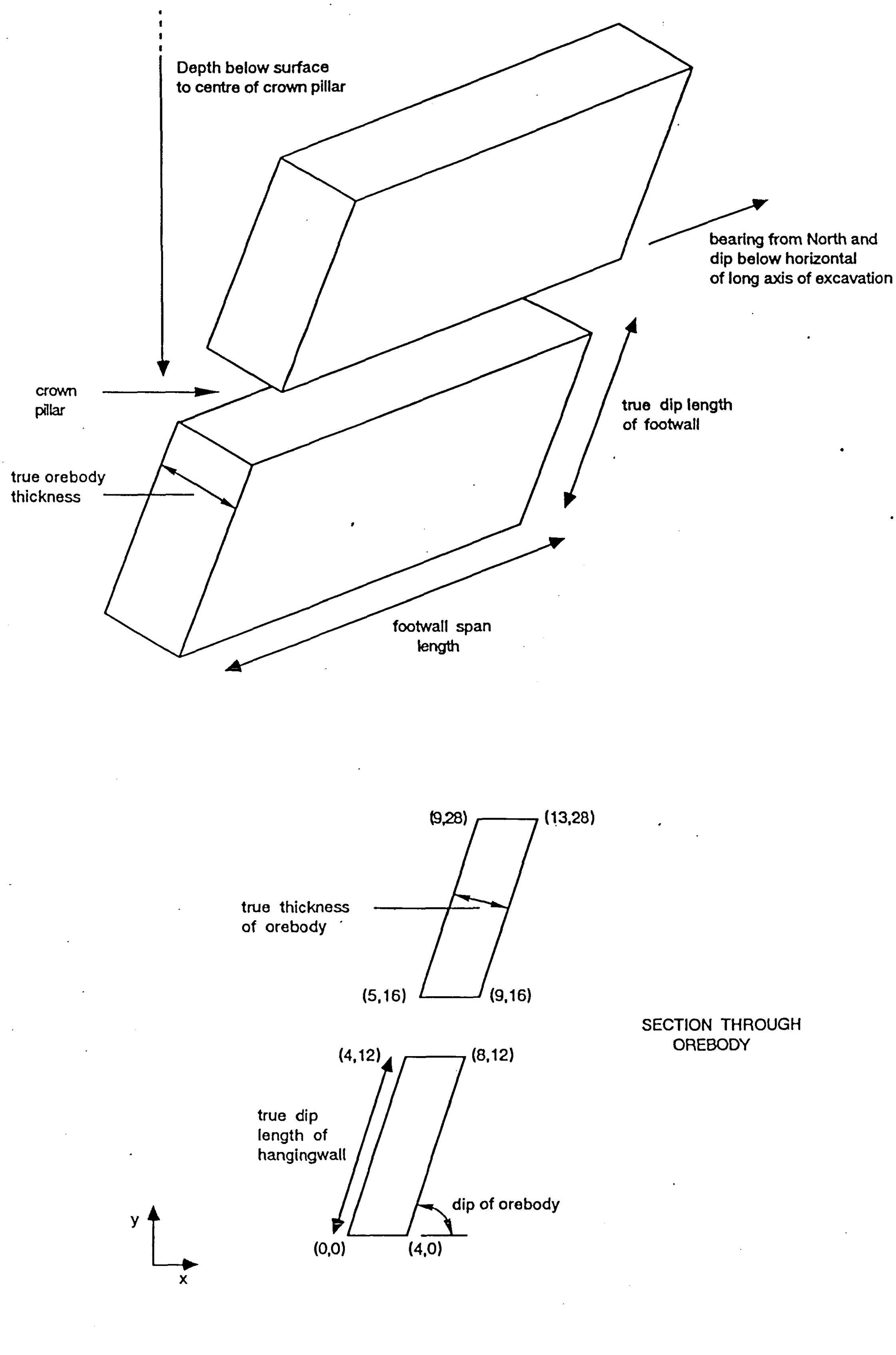


Figure 6.6 Basic geometric data required to use the developed knowledge-based system for crown pillar design.

are requested. These data are then used to determine a conceptual model of the rock mass (Section 6.3.2). These data include the number of joint sets present, the joint set spacings, the orebody thickness, the strike span of the hangingwalls of the stopes above and below the pillar, the true dip lengths of these hangingwalls and the depth below ground surface of the centre of the crown pillar. Figures 6.7 and 6.8 show examples of two of these questions and Figure 6.9 shows an example conclusion.

At this point the knowledge-based system has defined whether the rock mass can be considered to exhibit intact, jointed or discrete behaviour, and whether the problem is two or three-dimensional in nature (Figure 6.10). The system then asks for the *in situ* stress field in the vicinity of the crown pillar in an attempt to determine whether a complete or simple plane strain program should be used (Section 6.3.3). The data can be entered manually or can be accessed from a database directly. The *in situ* stress data are stored on a dBase 3 database file, which is an industry standard micro-computer database program. The database is searched using two location identifiers, only one of which has to be given, and the data are returned in the form of equations for the variation of magnitude of the three principal stresses with depth, together with their orientations. Using the depth of the centre of the crown pillar, which was previously elicited from the user, the actual *in situ* stress magnitudes at the crown pillar horizon can be determined. Figures 6.11, 6.12, 6.13 and 6.14 show screens of the *in situ* stress retrieval process.

The dip and bearing of the long axis of the open stopes are then requested by the system. This orientation is compared to the orientation of the *in situ* stresses, using a developed external PASCAL program, in order to determine whether the long axis of the excavation lies at an angle greater than 20° to an *in situ* stress direction. Thus, the system can determine whether complete or plane strain analysis is necessary (Section 2.3.2). Once the most suitable type of analysis has been determined, then the modelling aid part of the knowledge-based system is complete.

The rest of the system is comprised of the intelligent front-ends to the programs BEM11 and TWOFS, the structure of which were described in Section 6.3.4. The intelligent front-ends first enter into a dialogue with the user in order to determine a failure criterion for the crown pillar. The Hoek-Brown failure criterion

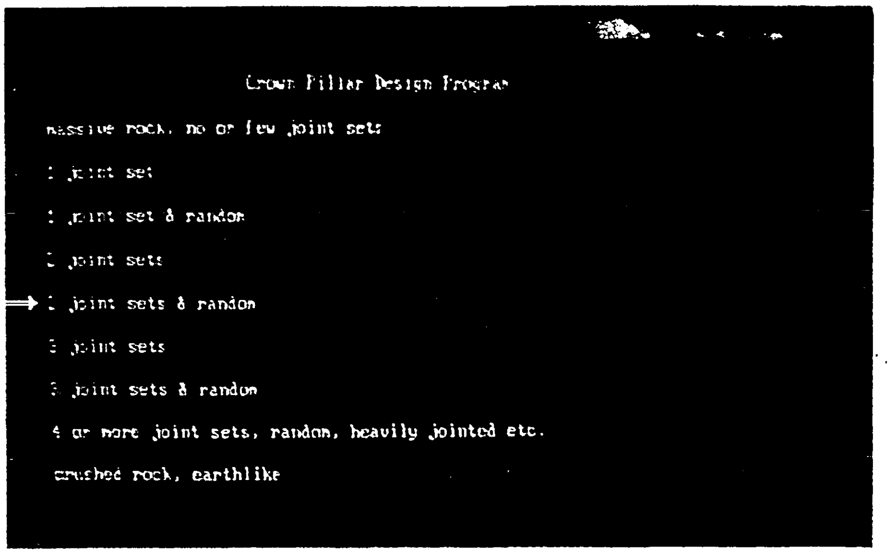


Figure 6.7 The user is asked to describe the number of joint sets present. The user moves the cursor (indicated by the arrow) to the description of the jointing that matches that of the rock mass, and presses the enter key.

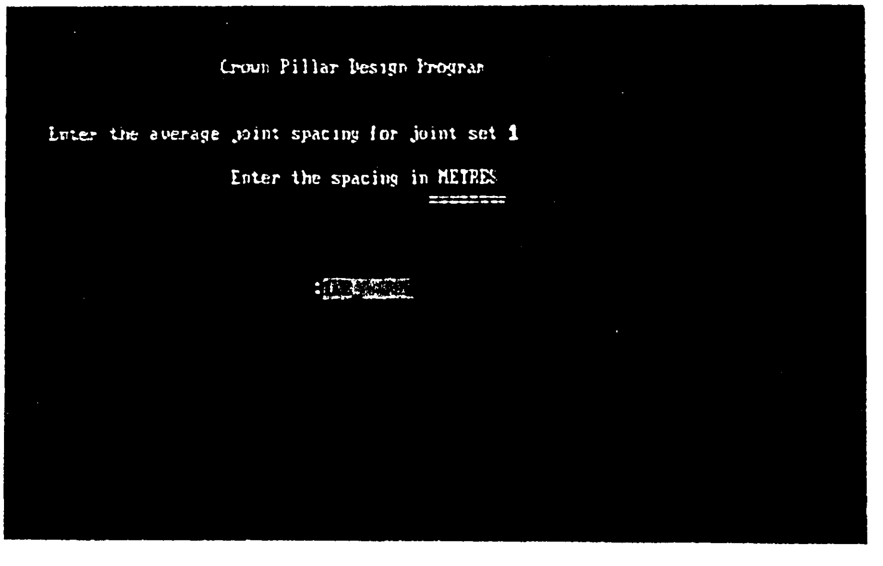


Figure 6.8 For each joint set the user is asked to enter the joint set spacing.

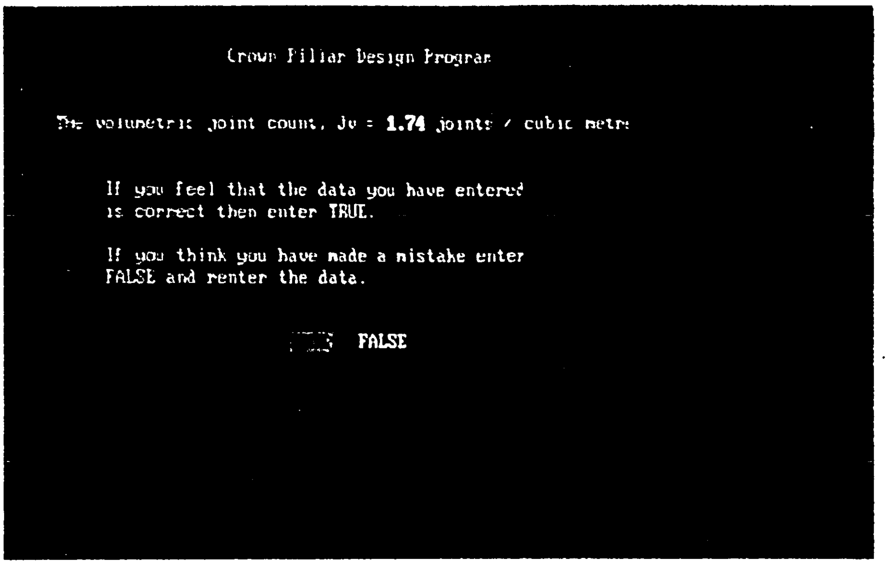


Figure 6.9 The system summarises the joint data that have been entered, and asks the user whether they are correct. If the data are not correct, the user presses FALSE, and all the joint data can be re-entered.

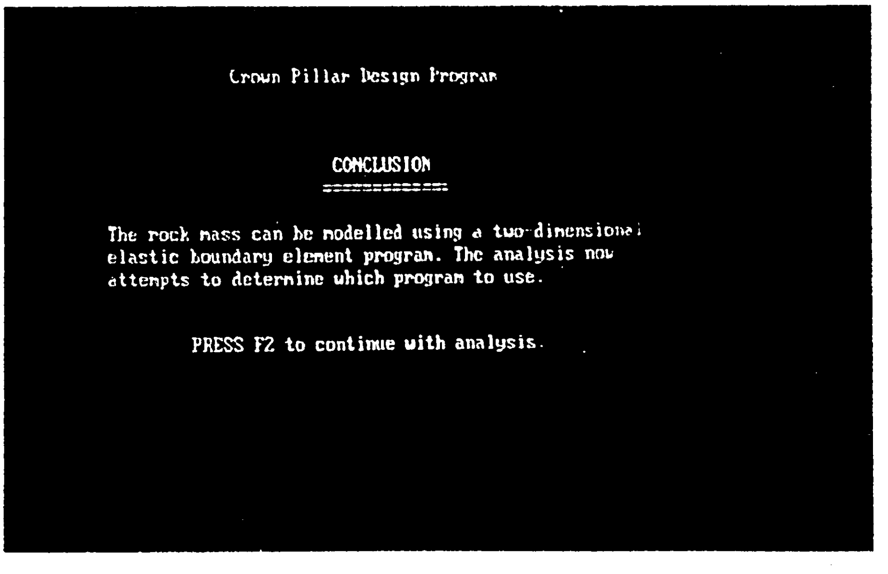


Figure 6.10 The system has evaluated the rock mass and conceptualised its behaviour, and together with the geometry data has decided that two-dimensional boundary element analysis is appropriate.

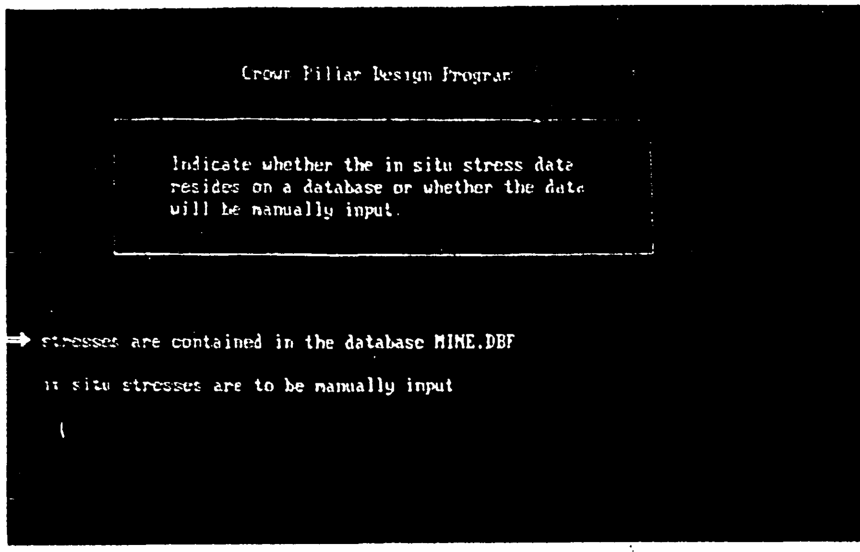


Figure 6.11 To determine the specific numerical analysis program required the *in situ* stresses for the problem are required. The stresses can be entered manually or retrieved from a database. The user moves the cursor and selects the required entry method.

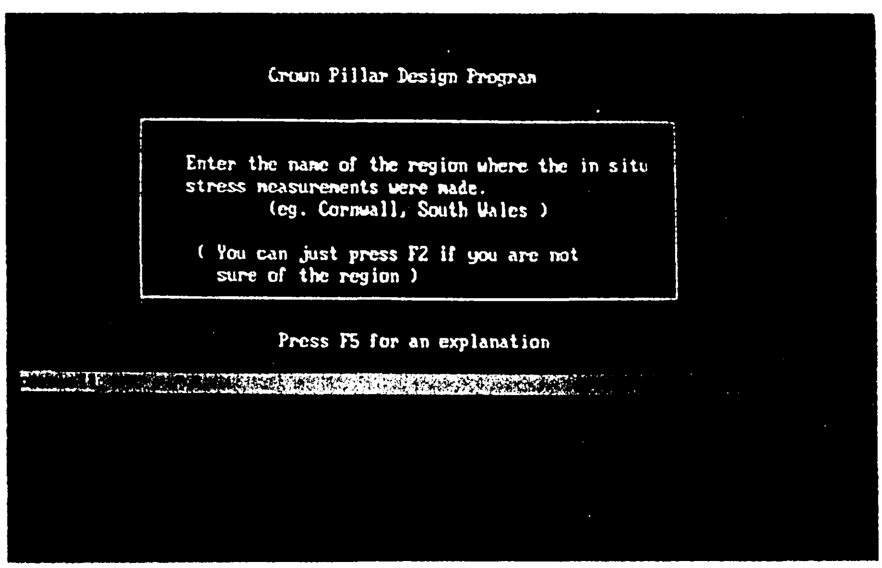


Figure 6.12 For the database retrieval of *in situ* stresses two search location descriptors are required, one for the region and the other for the site of the measurements. Only one of the search descriptors has to be given, and the user can press Function Key 2 if they are not sure of a descriptor. An explanation about the search descriptors is available by pressing Function Key 5 (see Figure 6.13).

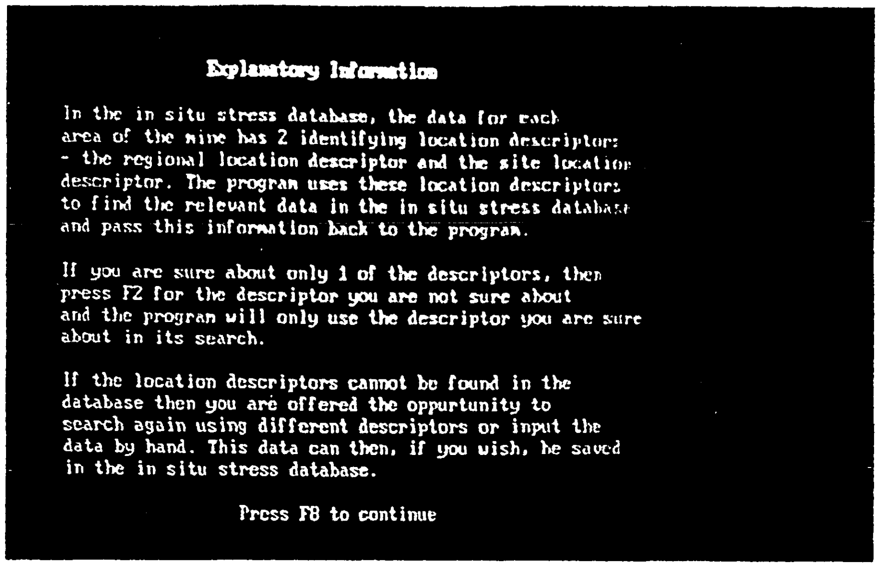


Figure 6.13 Explanation about the location descriptors for the *in situ* stress database. Pressing Function Key 8 returns the user to where they were in the program.

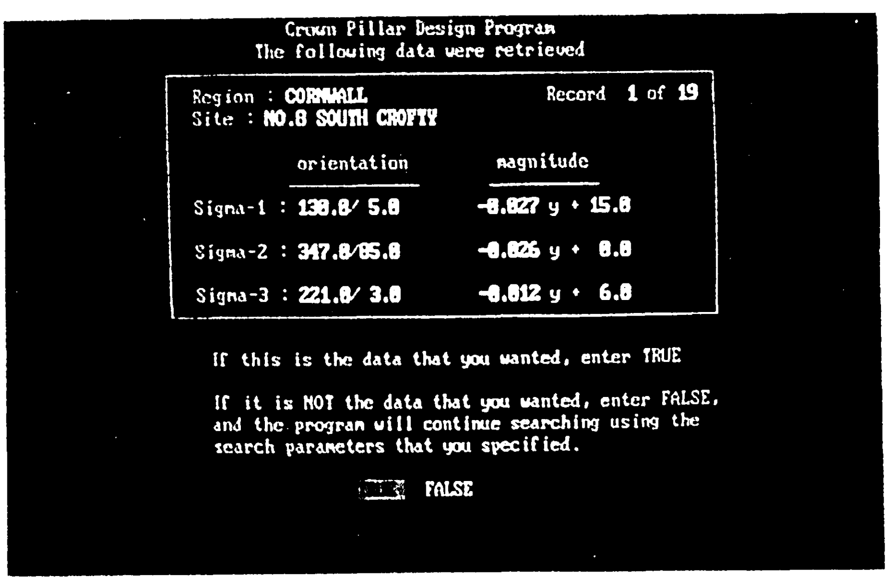


Figure 6.14 A data record from the *in situ* stress database. If these are the correct data then the user presses TRUE. Otherwise, by pressing FALSE, the system will search the database for further matches with the search descriptors. If there are no matches, then the system prompts the user to use different search descriptors, and also gives the user the option of inputting the data manually.

is used, and the program conducts a CSIR rock mass classification to determine the m and s constants of the criterion. Rock mass properties of the crown pillar are then either entered manually or retrieved from a database, using two search parameters – one for rock type, the other for rock location – in a similar way to that described above for the retrieval of the *in situ* stresses. Figure 6.15 shows an example of some material properties retrieved from the database.

The knowledge-based system then asks the user to discretise the boundaries of the stopes, with explanations available online about how a segment is divided into elements, and the co-ordinate system employed in the program (Figure 6.16). When the stopes have been fully discretised, then the points where stresses and displacements are to be calculated are inputted. This is in the form of a grid, whereby the bottom left-hand and top right-hand co-ordinates of the grid, and the grid spacings in the x and y directions, are specified (Figure 6.17 and 6.18). Other points of interest can also be specified along extra lines if desired. Finally, a summary of the grid information is given (Figure 6.19) and if it is correct then the crown pillar analysis data are summarised (Figure 6.20). Otherwise, all the internal points data can be re-entered.

6.3.6 Conclusions

In this section the underlying principles, knowledge and heuristics contained in the developed knowledge-based system were described. The prototype system contains sufficient knowledge to enable a mining engineer, who is a non-expert numerical modeller and user of numerical programs, to conduct numerical analysis of a crown pillar, that is correct from a modelling viewpoint, as well as being accurate in relating site conditions to those contained in the model. The system as it stands is by no means complete, and future work should be directed towards enlarging the knowledge base to incorporate subdivisions of the conceptual models, as well as other front-ends to numerical modelling programs.

The aim when building the system was to incorporate crown pillar design methodology into a modelling aid computer program, and hence to demonstrate the potential use of knowledge-based systems as a means of encoding design methodology for engineering rock mechanics. A small task domain was chosen for the system to be developed, so that the system could be built relatively quickly, but which at the same time could be investigated in detail. However, the basic

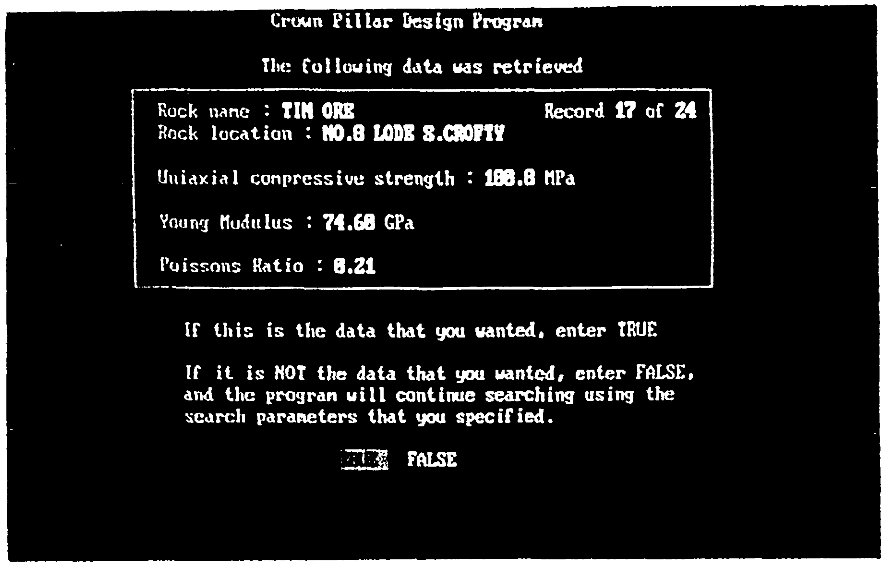


Figure 6.15 An example data record from the material properties database. The search mechanism for the material properties database is identical to that for the *in situ* stress database (see Figure 6.14).

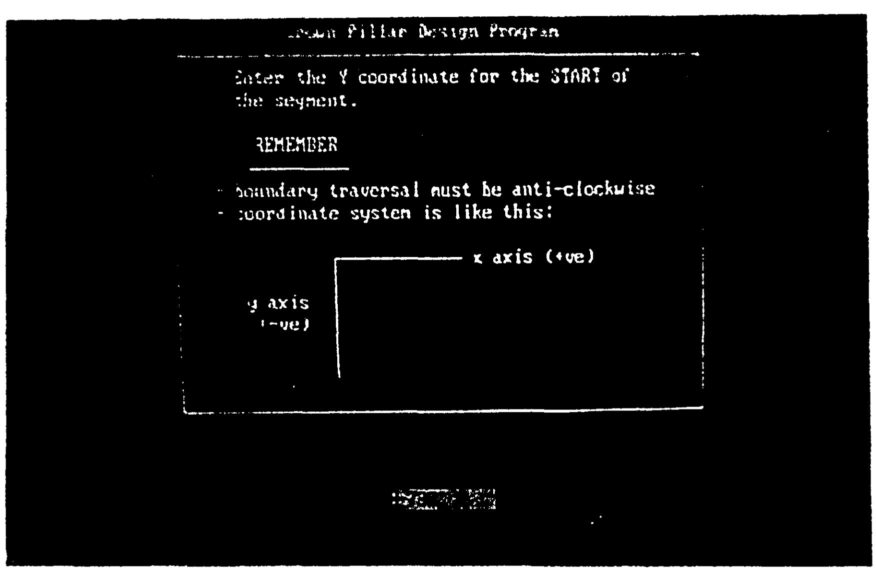


Figure 6.16 The boundaries of the stopes above and below the crown pillar have to be discretised – that is, divided into segments, which are themselves subdivided into boundary elements. The start and end x and y coordinates of each boundary segment are inputted, together with the number of elements along each segment.

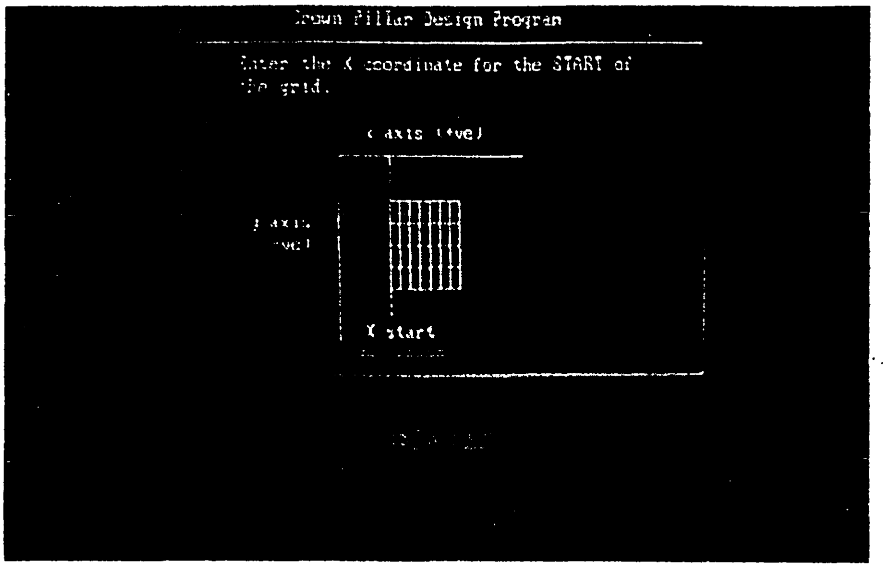


Figure 6.17 The internal points, where stresses and displacements are to be calculated, are determined by the intersections of a grid. The x and y co-ordinates of the bottom left-hand and top right-hand corners of the grid have to be entered. A diagram of the program co-ordinate system is given.

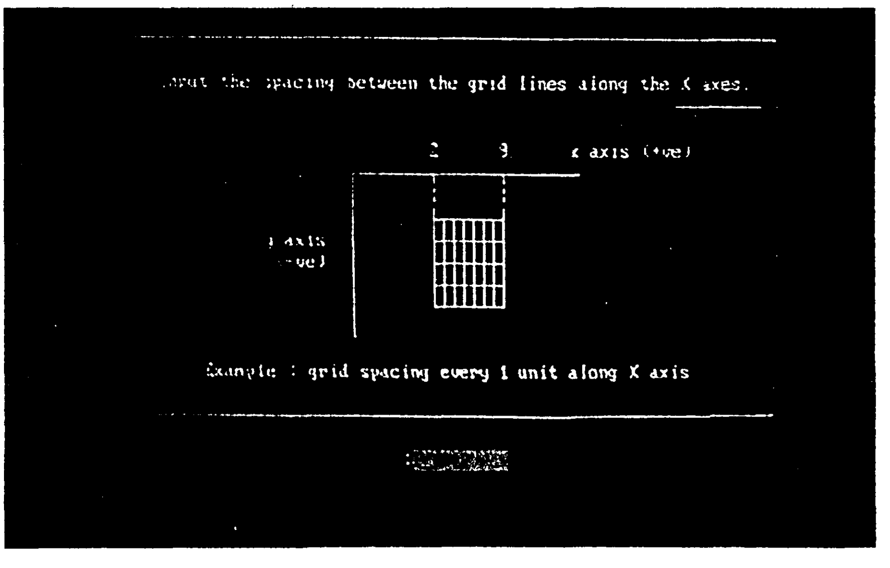


Figure 6.18 The spacing of the grid in the x and y direction is requested by the system. An example grid layout is given.

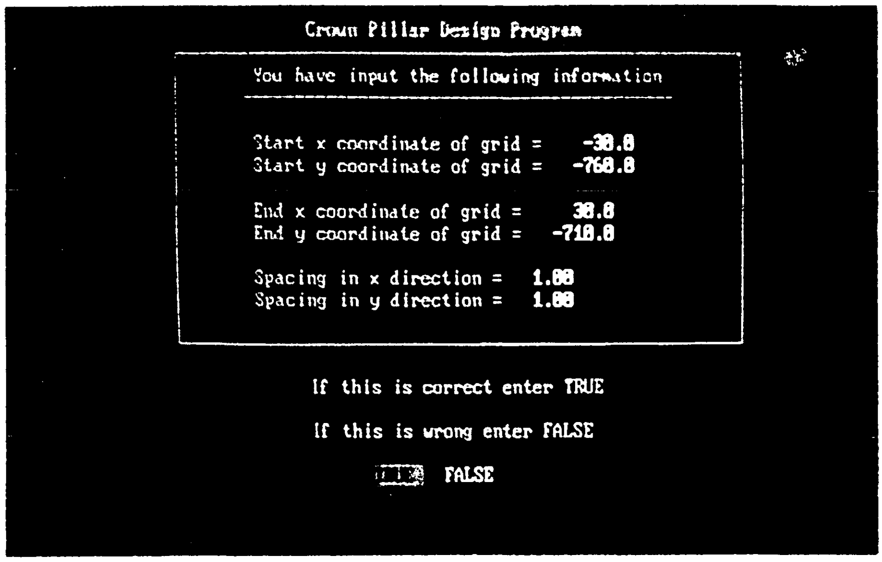


Figure 6.19 A summary of the internal grid point data that the user has inputted are given. The user presses TRUE if the data are correct. The user presses FALSE if the data are incorrect, and all the internal point data can be re-entered.

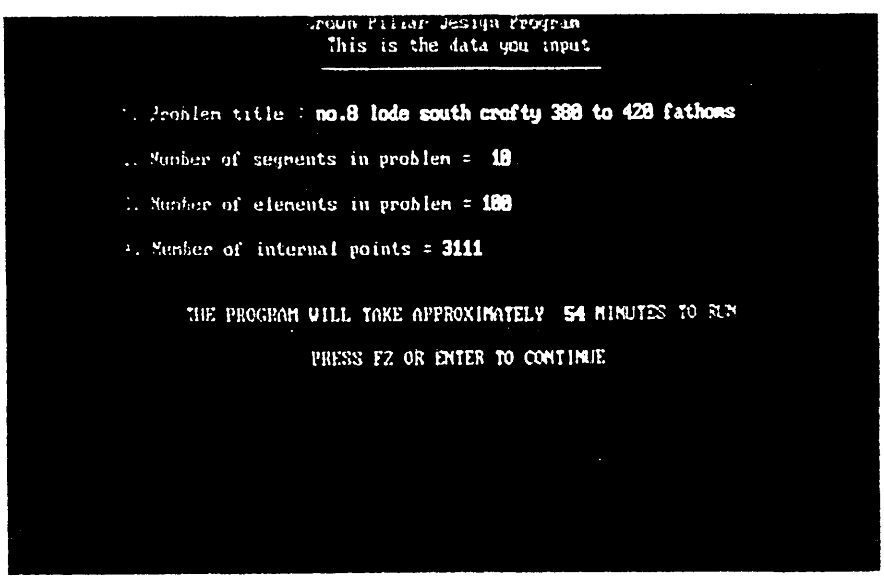


Figure 6.20 A short summary of the given crown pillar design problem is displayed, detailing the number of boundary elements used, and the number of internal points defined. The approximate length of time that the stress analysis will take is also given.

structure of the system was designed in such a way that it could be expanded to incorporate more task domains, and different types of analysis, without altering the original structure.

6.4 Conclusions and Future Improvements

The knowledge-based system that has been developed is only a prototype at present and requires further development if it is to be used in practice. In addition, a wider range of intelligent front-ends and associated numerical programs are needed, in order that most encountered situations accounted for. Furthermore, these intelligent front-ends need to become more automated, especially with regard to stope discretisation and internal point location, which are still relatively slow and prone to typing errors (although these can be identified in the summaries and the data re-entered). Incorporating a digitiser interface so that data could be taken directly from vertical cross-sections and longitudinal sections would be a solution to this problem.

For the future, if the developed modelling aid for numerical analysis is to be incorporated into mine design knowledge-based systems, as was envisaged in the systems described in Section 1.3.2, then the simple rule-based approach adopted for the research would be too restrictive. Frame-based representation schemes (Section 5.2.2) allow the representation of complex objects and their associations, and at the same time can incorporate rule-like structures. A blackboard approach (Nii, 1986), for the communication between independent modules, can then be used in which, once the blackboard has been specified, the independent modules can be developed separately.

CHAPTER 7
CASE STUDY OF
NO.8 OREBODY SOUTH CROFTY

7.1 Introduction

This chapter presents a case study of open stope and pillar design in a narrow, steeply dipping, tabular orebody, using the implemented boundary element programs (Section 2.3.5), the developed open stope and pillar design methodologies (Sections 3.5 and 4.4, respectively) and the prototype knowledge-based system for crown pillar design (Section 6.3).

The No. 8 orebody at South Crofty tin mine, Cornwall, U.K. was selected as the site for the design exercise. The rock mass properties (Randall, 1987) and the *in situ* stress field (Pine *et al.*, 1983 a,b; Batchelor and Pine, 1986) had been previously determined. An investigation of the rock structure in the vicinity of the No. 8 orebody was conducted in order to determine the orientations of the main joint sets present, and their spacings. Using these rock property data and the Hoek-Brown empirical strength criterion, a rock mass strength criterion for the No. 8 orebody was determined.

In Section 7.2, a full description of South Crofty mine is given and a summary of the geology, rock mass properties, rock structure and *in situ* stresses present, is given. In Section 7.3, the design problem for the No. 8 orebody is defined and then, using the stope and pillar design methodologies developed in Sections 3.5 and 4.4 respectively, a series of two and three-dimensional stress analyses are conducted. The developed knowledge-based system for crown pillar design (Section 6.3) is also shown to be of use, in determining the correct type of numerical program to use and in speeding up, simplifying, and eradicating unnecessary mistakes in their data input. The results from these analyses are presented in Section 7.4 and finally, in Section 7.5, the conclusions and practical implications are discussed.

7.2 South Crofty Mine

7.2.1 Mine Description

South Crofty mine is situated at Pool, between Camborne and Redruth, in Cornwall, U.K. (Figure 7.1). South Crofty is probably the oldest tin mine still at work in the U.K., although for several centuries its shallow workings exploited copper, rather than the deeper tin mineralisation. South Crofty is an amalgamation of twelve old mines, with orebodies (locally referred to as lodes) totalling approximately 4.5 *km* on strike.

The two main shafts, about 430 *m* apart, are Robinson's (683 *m* deep) and New Cook's Kitchen (732 *m* deep), each being timbered, and rectangular (4.6 *m* x 1.8 *m*) in cross-section. Two cages, for men and materials, are used in Robinson's shaft, whilst Cook's, the main ore raising shaft, has balanced 7.0 *tonne* capacity skips. Both these shafts are downcast (clean air entering the mine down them), and used air is exhausted via two refurbished shafts in adjoining old mines: Taylor's shaft to the East, and Roskear shaft to the West (Figure 7.2).

Robinson's and Cook's shaft give access to levels down to the 380 fathom level. From each level crosscuts are driven NNW or SSE, linking in to on-lode development which generally run ENE or WSW. Typically these drives are 2.4 *m* x 2.4 *m* in cross-section. Below the 380 fathom level, access is gained by a sub-decline shaft, which serves the 400 fathom level (730 *m* below surface), the 420 fathom level (770 *m* below surface), and is being extended down to the 445 fathom level (820 *m* below surface). This sub-decline shaft dips at 1 in 4, is 4 *m* x 3 *m* in cross-section, and is equipped with conveyor belt haulage.

7.2.2 Mining Methods

The mining methods at South Crofty are influenced by the erratic nature of the ore distribution, the kaolinisation of the wall rocks, and the orebody width. In wide (+2 *m*) lodes the main method used is sublevel longhole open stoping. In narrower lodes, overhand shrinkage stoping is usually employed. Until recently, longhole and shrinkage stoping each accounted for about 40% of the ore production, the remaining 20% of ore production arising from stope development. Longhole stoping now accounts for 55% of the ore production, due to refinements in the longhole technique, and the application of longhole stoping to narrower lodes (−2 *m*). Longhole

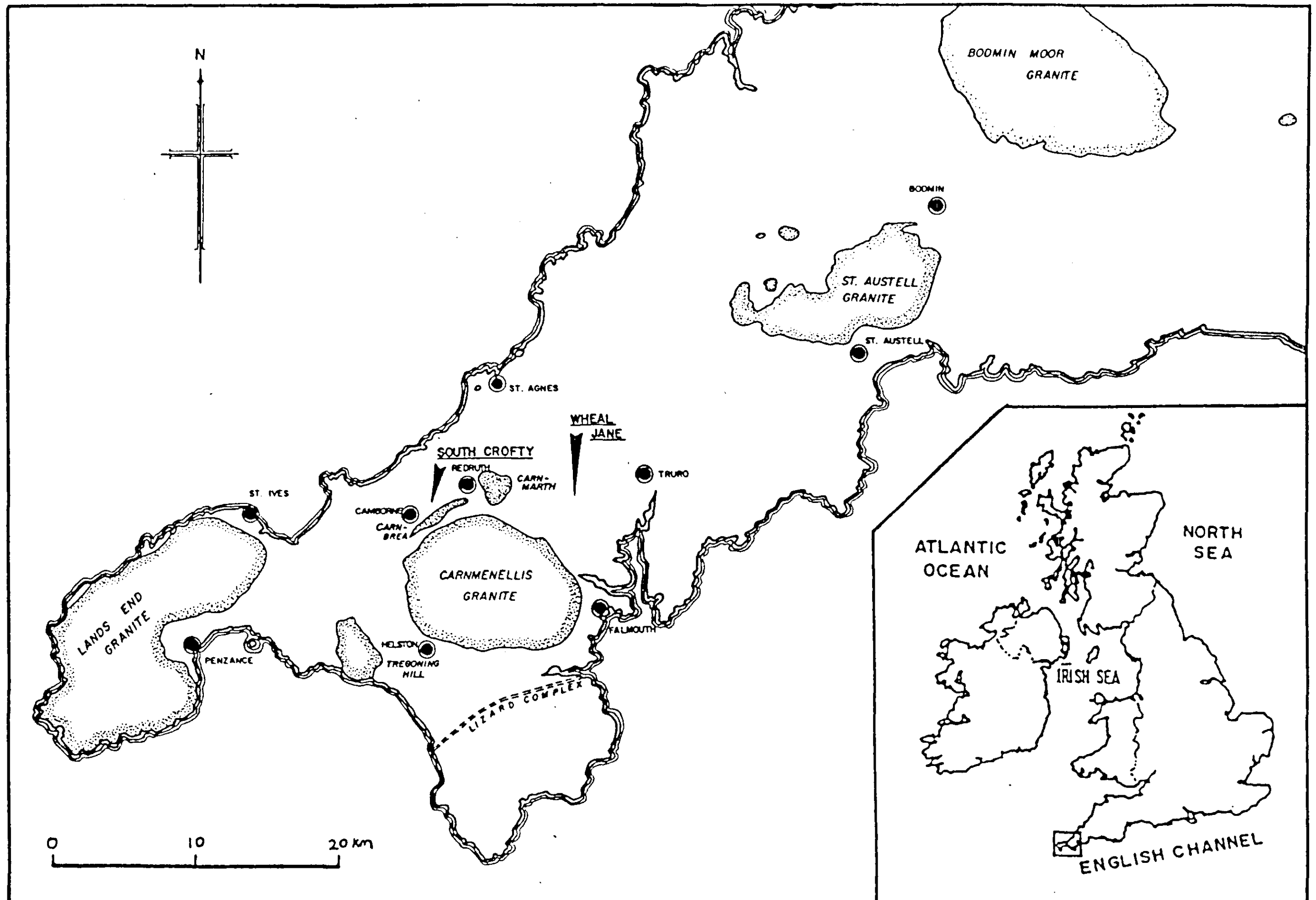


Figure 7.1 Location of South Crofty mine.

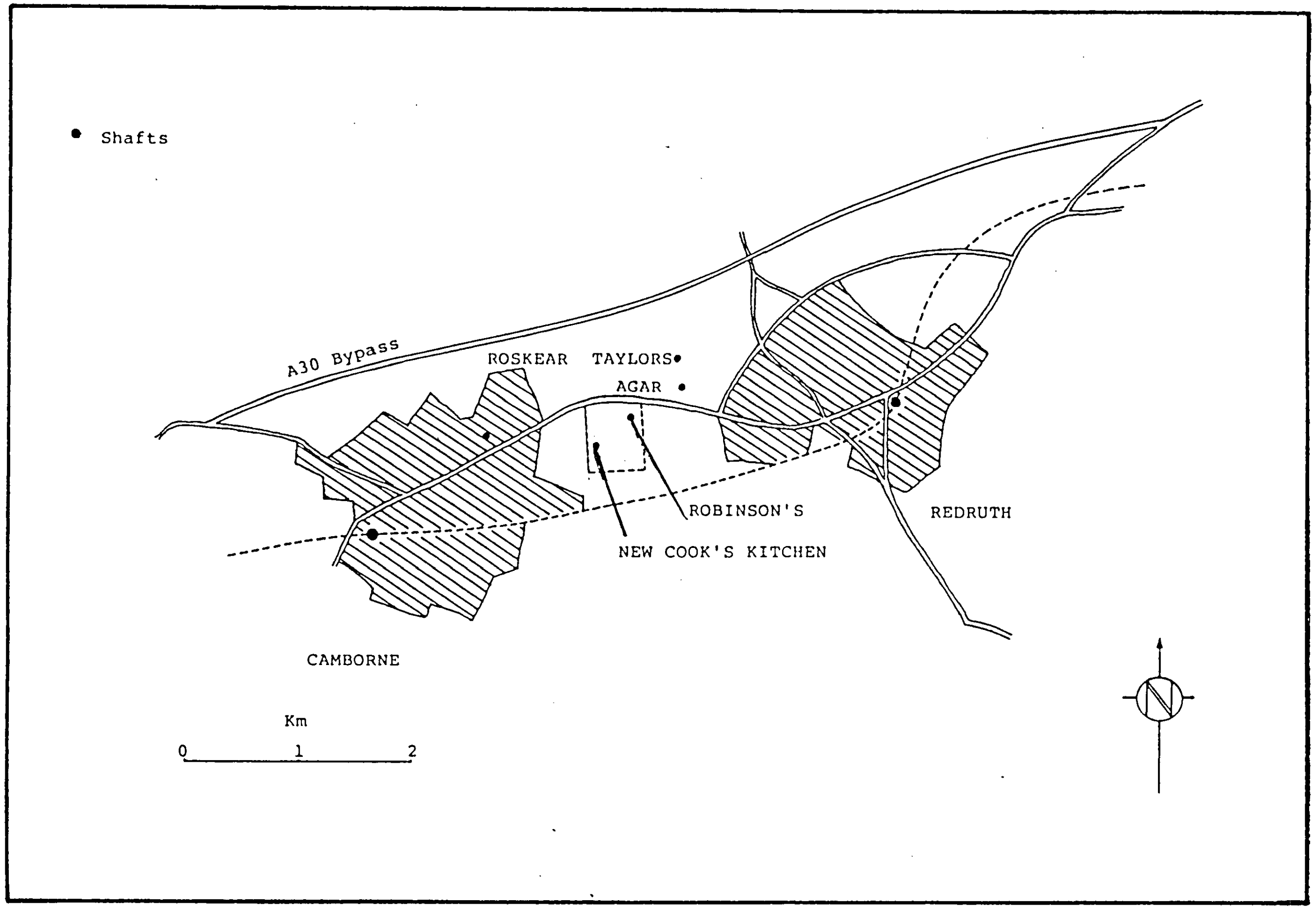


Figure 7.2 Location of shafts at South Crofty mine.

stoping has been found to be both safer and physically easier for the miner, than shrinkage stoping, whilst at the same time longhole stoping has been found to have a higher productivity and lower costs.

In Figure 7.3 the sublevel longhole open stoping method used at South Crofty is given. Longhole stoping proceeds by developing a lode drive and footwall haulage drive, with connecting drawpoints. Raises are then driven on-lode for ore grade sampling purposes. When an economic mining block has been delineated then a sublevel is driven in-between the two main levels which define the top and bottom of the mining block. Main levels are positioned at approximately 37 m vertical intervals. Longhole drilling is then conducted in the lode drives and sublevels. In the lower level lode drive, drilling is upwards within the orebody. In the upper level lode drive, drilling is downwards within the orebody if no crown pillar is to be left below this lode drive, otherwise drilling is not conducted. In the sublevel, drilling is both upwards and downwards within the orebody. At South Crofty fan drilling is employed. A stope is begun by widening a raise to produce a cut-off slot and the orebody is undercut, by blasting first the ore that was drilled from the lode drive on the lower level. Mining proceeds by retreating, with blasting from the sublevel and upper level being staggered, to utilise the two free faces created by the undercut, and the cut-off slot.

Traditionally, haulage drives were on-lode, with ore drawn from stopes by 'Cousin Jack' chutes. Chutes are now being replaced by drawpoints, at approximately 7 m centres, with the haulage drive being driven in the footwall, approximately 7 m from the stope. Drawpoints have proved to be far more efficient than chutes as a means of removing ore from stopes.

7.2.3 Geology

The South Crofty workings lie mainly in the Carn Brae granite which is the northern ridge of the Carnmenellis granite boss. The granite outcrops to the south of the mine as a prominent NE–SW trending ridge called Carn Brae (Figure 7.1). Metamorphosed sediments and volcanics, known locally as killas, also lie within the metamorphic aureole of the granite and exhibit varying degrees of alteration. Both the granite and the killas are cut by later quartz-feldspar biotite porphyry dykes. These are characteristically steeply north dipping, strike approximately ENE, and are known locally as elvans.

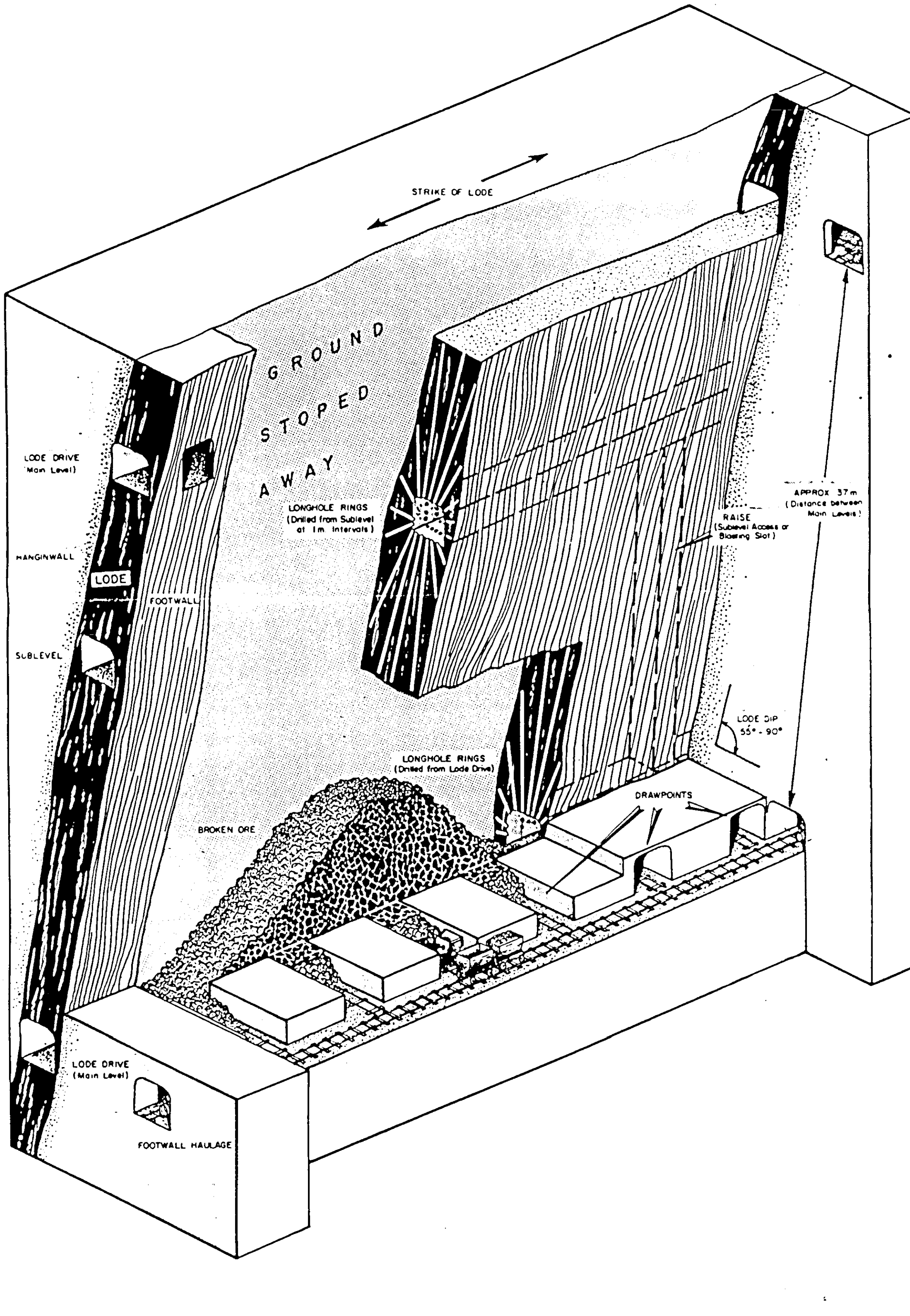


Figure 7.3 Longhole open stoping method used at South Crofty.

The present South Crofty workings are almost exclusively in granite. The mineralisation is associated with two sets of structures intersecting almost at right angles.

Set 1

An ENE striking series. There are over 30 such lodes in the ore reserve and they contain the majority of the economic mineralisation present at South Crofty. These lodes dip North and South at angles greater than 50° to the horizontal. The lodes are hydrothermal in nature and contain minerals such as cassiterite, wolframite and chalcopyrite. Gangue minerals include chlorite, quartz, tourmaline, haematite and arsenopyrite. The proportion of all minerals varies from lode to lode, and within lodes. The area is regarded as classical in demonstrating the zoning of minerals, many of the mines in the area having started life as copper mines and then changed to tin mines at depth, when cassiterite become the dominant ore mineral.

Set 2

The NNW striking series. At South Crofty these lodes are economically barren. They dip steeply in both directions and cut and fault the ENE striking lodes, and are known locally as crosscourses. The crosscourse infilling is predominantly quartz with some fluorites, haematite, pyrite and chalcopyrite.

Both sets of lodes are filled fault fissures with varying degrees of wallrock alteration and/or replacement. In addition, a third mineralisation set exists. Swarms of early sub-horizontal quartz-feldspar veins with minor quantities of wolframite and arsenopyrite host a later phase of pervasive cassiterite replacement mineralisation.

The majority of the workings lie to the North of Robinson's and New Cook's Kitchen shafts and these lodes are normally narrow (0.5 m – 1.5 m wide) and conform to the description of Set 1 above. Tin distribution at South Crofty, unlike other mines in Cornwall, is erratic. Tin values occur in lenses, some of which may be quite large and extend over several levels for hundreds of metres on strike, whilst others may be as small as 20 metres square. Average values in ore reserve blocks can run as high as 4.00% Sn (tin metal) or as low as 0.7% Sn (minimum stoping block cut-off grade). The cut-off between payable ore and unpayable ore is normally quite sharp.

The most important lodes east of the Great Crosscourse (Figures 7.4, 7.5 and 7.6) are the No. 2, 4, 8, and potentially, Providence lodes. The very high payability of the No. 2, 4, and 8 lodes over an approximately 400 metre strike length, results in them accounting for approximately 21% of the current demonstrated ore reserve by tonnage. The No. 8 lode is the widest of these lodes (averaging 2.5 m) with an average ore grade of 2.4% Sn. The No. 8 lode dips around 70° South above the 380 fathom level, but around the 420 fathom level it flattens out to around 55° and narrows to around 1.7 m true width. The No. 8 orebody can be seen to be one of the major ore production centres of the mine, and will continue to be so for some time. Consequently, great importance is attached to the planning and design of new stopes in this orebody below the present mining level (400 fathom level), and to the maximisation of ore recovery.

7.2.4 Geotechnical Data

In Situ Stress

The *in situ* stress field in the Carnmenellis granite has been fully investigated (Pine *et al.*, 1983a,b; Batchelor and Pine, 1986). The accumulated data has enabled the orientation of the *in situ* stresses to be determined (Table 7.1), as well as the variation of the magnitude of the *in situ* stresses with depth to be determined (equation 7.1).

$$
\begin{aligned}
\sigma_1 &= 15 + 0.0275 \quad (MPa) \\
\sigma_2 &= 0.026H \quad (MPa) \\
\sigma_3 &= 6 + 0.0118H \quad (MPa)
\end{aligned}
\qquad (7.1)
$$

where H = depth below surface in metres.

Rock Mass Properties

The main lithologies in the vicinity of the No. 8 lode are the granite of the hangingwall and footwall, and the orebody rock. Both the orebody and adjacent granite and generally competent but there are some localised zones of kaolinisation within the granite. No *in situ* rock mass properties have been measured but laboratory tests on the hangingwall, footwall, and orebody rock have been conducted (Ran-

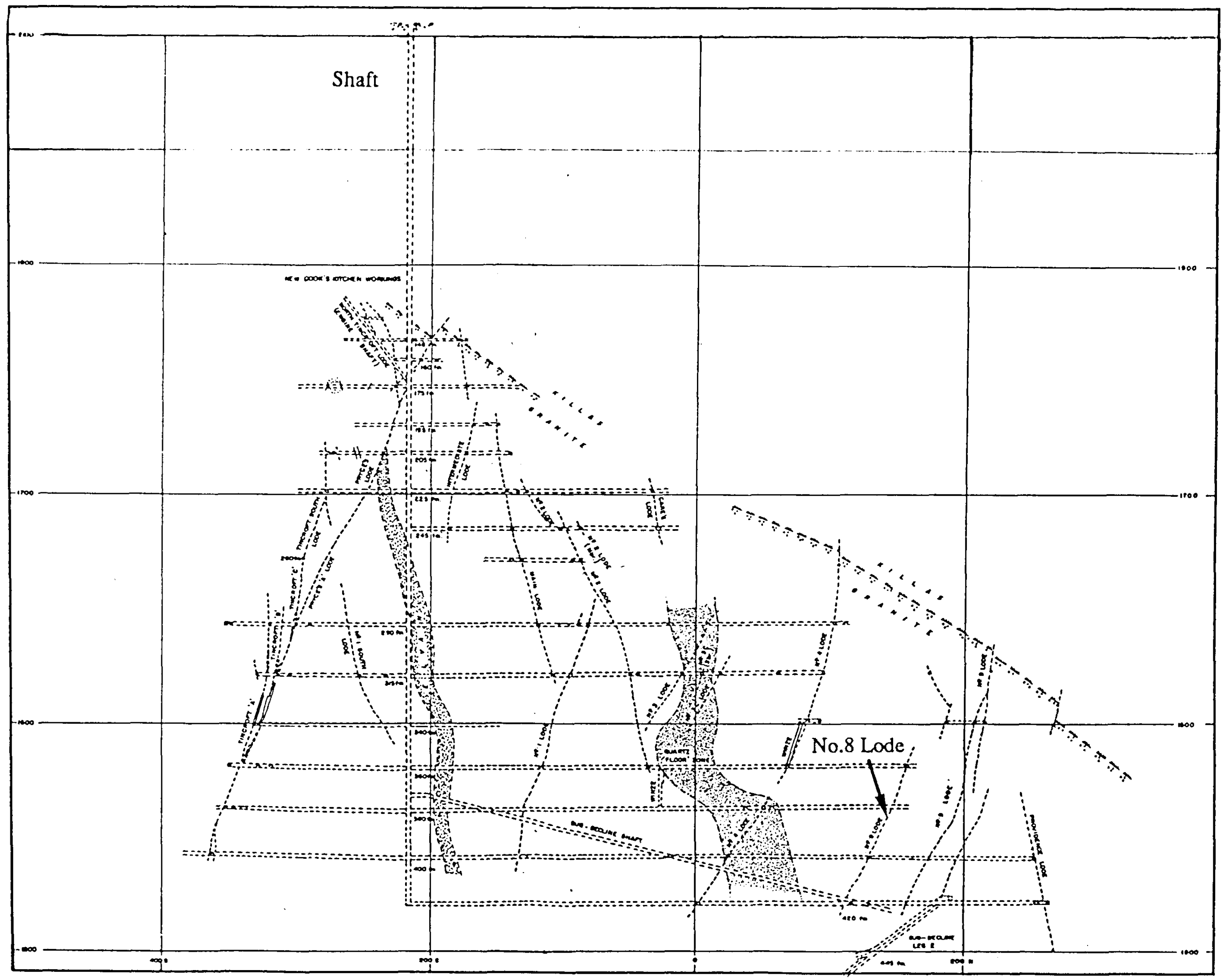


Figure 7.4 Geological transverse section of South Crofty.

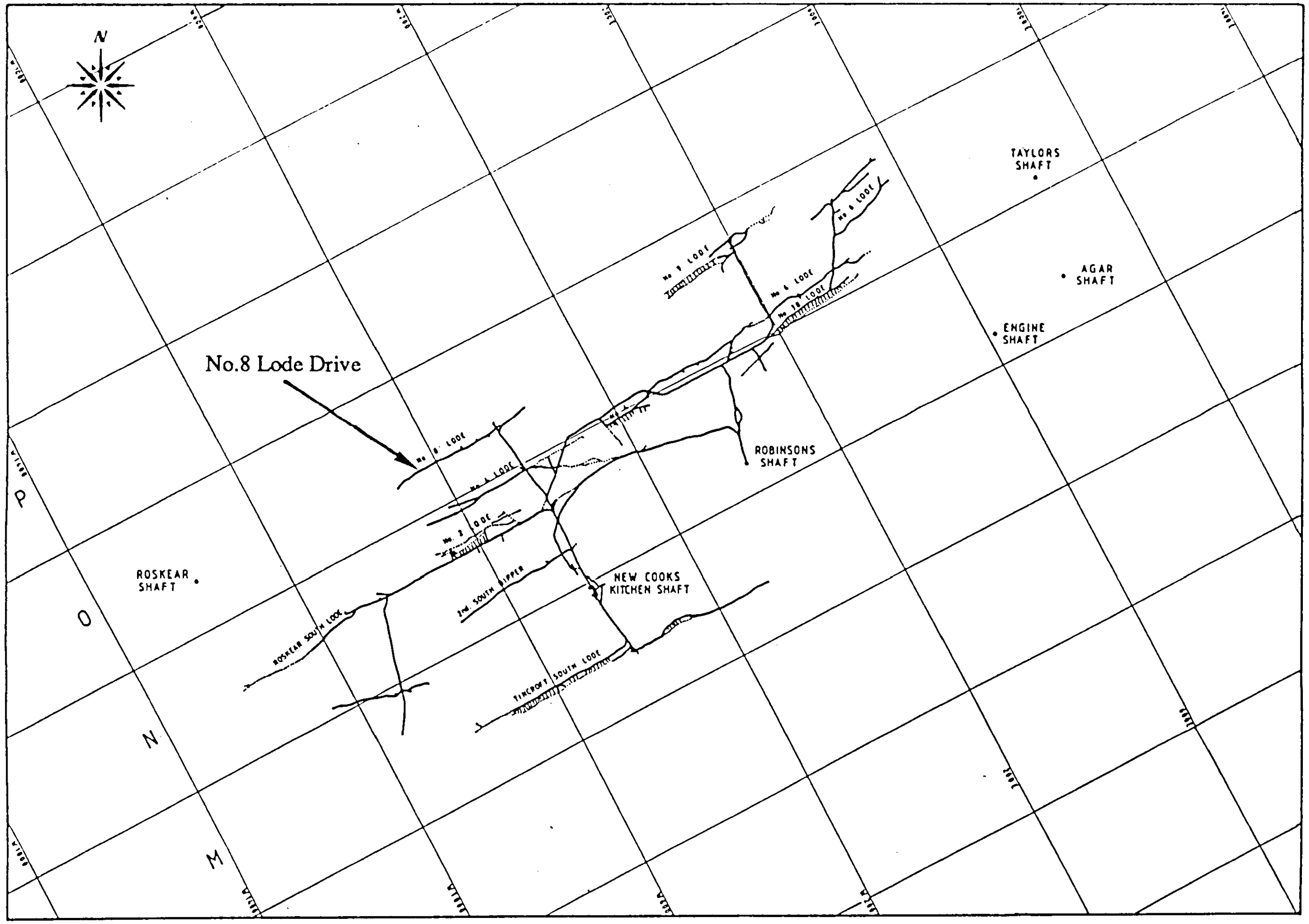


Figure 7.5 380 fathom level plan at South Crofty.

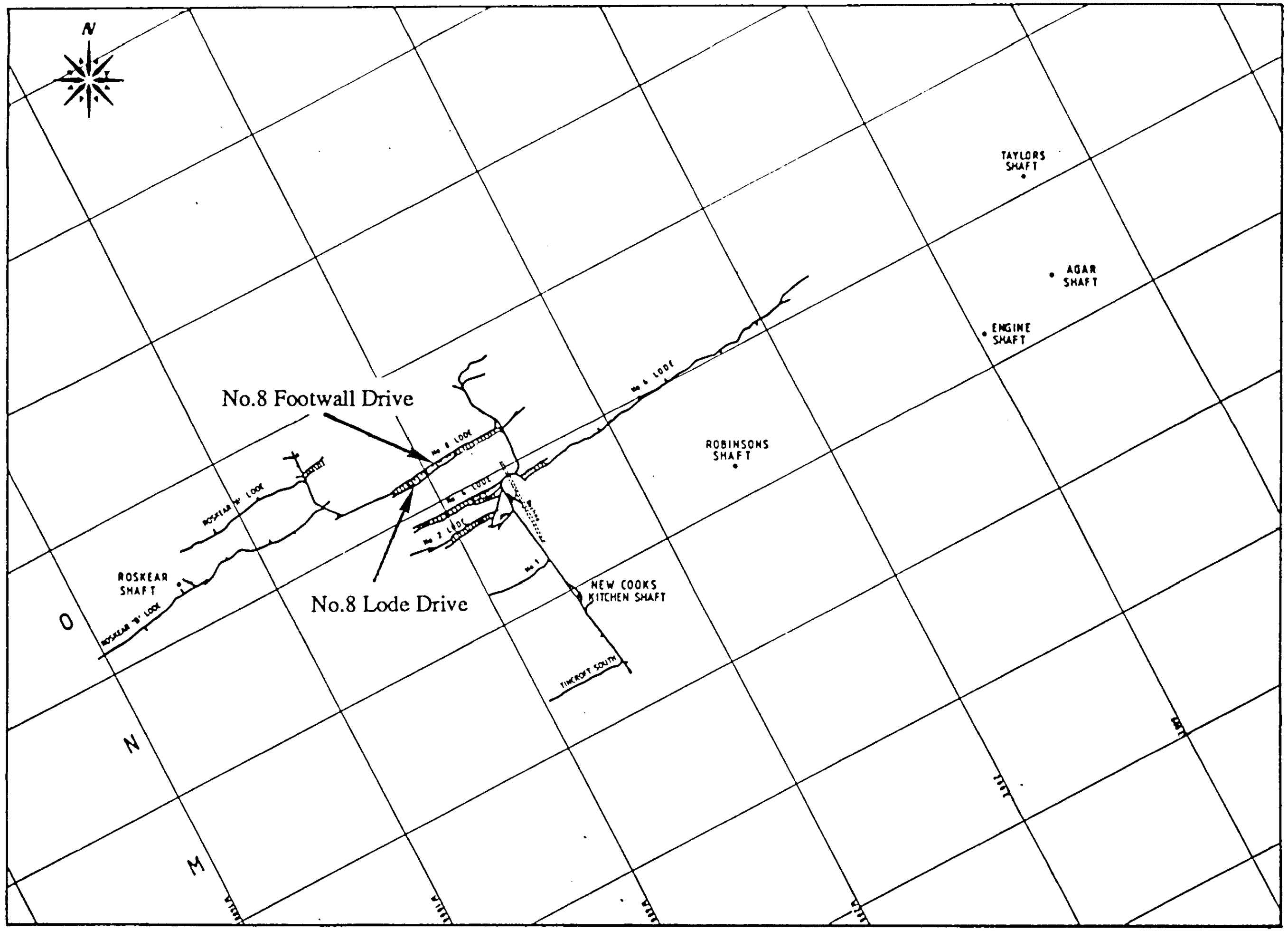


Figure 7.6 400 fathom level plan at South Crofty.

Stress	Orientation	Dip
σ_1	130°	5°
σ_2	347°	84°
σ_3	221°	3°

Table 7.1 Orientation of the *in situ* stresses at South Crofty. Orientations are bearings from True North and all dips are positive below the horizontal.

Rock	$UCS\ (MPa)$	$E\ (GPa)$	ν
Hangingwall Granite	190 (190)	65 (65)	0.21 (0.21)
No. 8 Orebody Rock	180 (190)	70 (65)	0.21 (0.21)
Footwall Granite	190 (190)	65 (65)	0.21 (0.21)

Table 7.2 Material properties of the rocks in the vicinity of the No. 8 orebody. Values in brackets are those used in the numerical analyses.

Joint Set	Strike (^{0})	Dip (^{0})	Spacing (m)
1	060 - 240	65^0 South	7
2	030 - 210	65^0 North	3
3	060 - 240	90^0	5
4	150 - 330	90^0	4

Table 7.3 Joint set data in the vicinity of the No.8 orebody.

dall, 1987). The mean uniaxial compressive strength for the footwall and hangingwall granite is around 190 MPa with a standard deviation of 30 MPa. These strengths are higher than those reported for Carnmenellis granite from quarries, where the range of strength is typically 100 to 150 MPa. However, Carnmenellis granite cores from depths of 1400 to 2000 m from the tests of Batchelor and Pine (1986), had a uniaxial compressive strength of 190 MPa.

There have been relatively few compression tests conducted on the ore, but Randall (1987) reported that cores from the No. 8 orebody had a uniaxial compressive strength of 180 MPa (with a standard deviation of 45 MPa). Due to the limited number of tests conducted on orebody rock, the No. 8 orebody and adjacent granite hangingwall and footwall are assumed to have the same uniaxial compressive strength for design purposes.

The Young's modulus for the granite and the No. 8 orebody are also approximately equal, with the orebody being slightly stiffer. Randall (1987) measured the Young's modulus of the hangingwall and footwall granite to be approximately 65 GPa, which is the same value as that determined for Carnmenellis granite by Pine *et al.* (1983). The Young's modulus of the No. 8 orebody was measured to be approximately 70 GPa. For design purposes it is assumed that the intact Young's moduli for both the granite and the No. 8 orebody are 65 GPa. The measured Poisson's ratios of the granite and the No. 8 orebody were similar at approximately 0.21. The material properties of rocks in the vicinity of the No. 8 orebody are summarised in Table 7.2.

The local geological structure at South Crofty has been reported elsewhere (Pine *et al.*, 1983), but this has in general been restricted to exposures away from the orebodies. A scanline survey was therefore conducted in the vicinity of the No. 4 and No. 8 orebodies (Figures 7.5 and 7.6) on the 380 fathom and 400 fathom levels. In total 1.2 km of scanline were mapped in lode drives, footwall drives, crosscuts and drawpoints.

It was found that there were four distinct joint sets and in Table 7.3 the dip, strike directions and spacings of these joint sets are summarised. The two main sub-vertical joint sets, sets 3 and 4, strike parallel and perpendicular to the orebody, respectively. It was found that the spacing of these joint sets in the vicinity of the No. 4 orebody was similar to that in the vicinity of the No. 8 orebody, and similar to that reported by Pine *et al.* (1983). These joint sets were in general closed, although occasionally a thin coating of kaolinite is present. Few sub-horizontal joints were found, although a scanline of New Cook's Kitchen shaft gave a sub-horizontal joint set spacing of approximately 10 m. In the No. 8 orebody footwall drive an early sub-horizontal, quartz-feldspar vein is apparent.

Joint set 1 has a strike parallel to the No. 8 orebody, dips 65° to the South, and generally has tourmaline infilling. It was found that the spacing of this joint set became appreciably smaller (approximately 4 m compared to an average of 7 m) in the vicinity of the No. 4 hangingwall.

Joint set 2 is frequently seen along the No. 8 orebody footwall drive on the 400 fathom level and is generally closed. However, kaolinisation of rock adjacent to this joint set occurs, occasionally, and this results in a very low cohesive strength rock with the occurrence of running ground in some areas. Joint set 2 has a spacing of around 3 m in the vicinity of the No. 8 orebody, whereas near the No. 4 orebody the spacing increases to around 6 m.

Rock Mass Classification

The CSIR rock mass classification system due to Bieniawski (1976), as described by Hoek (1988) and given in Section 3.5.2, was used to develop a rock mass strength model for the No. 8 orebody. Hoek (1988) modified the original classification scheme slightly, for use in determining the m and s constants for the Hoek-Brown failure criterion (equation 3.6). In addition to the CSIR classification, an NGI classification, due to Barton *et al.* (1974), was undertaken on the No. 8 orebody, but with the last two factors of this classification (effect of water and effect of stress) ignored. This is in line with the description of the use of the NGI classification for stope stability studies given in Section 3.4.1. A summary of the application of these two classifications to the No. 8 orebody is given in Table 7.4. Since there were few exposures of the orebody, the rock mass classification for the hangingwall granite has been used for the orebody, since they appear to be be very similar.

The two rock mass classification systems can be compared using the equation:

$$RMR' = 9 \ln Q' + 44 \tag{7.2}$$

Inserting the determined value of Q' (17–33) in equation 7.2 gives a computed value of RMR' of 69–75, which is close to the determined value for RMR' of 82.

CSIR classification

Parameter	Description	Value
Strength	190 MPa	12
RQD (Joints / m^3)	5 Joints / m^3 = 100 %	20
Joint spacing	0.6 - 1.0 m	20
Joint condition	Continuous Hard joint wall rock Hard Impervious filling.	20
Groundwater	Completely dry	10
		Total = 82

NGI classification

Parameter	Description	Value
RQD (Joints / m^3)	5 Joints / m^3 ~100 %	100 %
Jn (Number of joint sets)	3 Joint sets	9
Jr (Joint Roughness)	Rough planar to irregular	1.5 - 3.0
Ja (Joint alteration)	Surface staining only	1.0
Jw (Presence of water)	~ Dry	1.0
SRF (Stress condition)	Not applicable - set to 1.0	1.0

$$Q' = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} = 17 - 33$$

Table 7.4 CSIR and NGI rock mass classifications of the No.8 orebody.

The rock mass ratings given in Table 7.4 are considered representative of average conditions. Strengths in kaolinised areas will be lower.

Using the range of RMR' values determined ($RMR' = 69, 75, 82$) and the equations relating RMR' with the Hoek-Brown parameters for undisturbed rock (equations 3.9 and 3.10), a range of m and s values for the failure criterion for the No. 8 orebody were determined (Table 7.5). Using the determined values of s, the associated uniaxial compressive strengths of the rock mass were calculated using equation (3.6b). Together with the m and s values for each value of RMR', the range of the Hoek-Brown failure criterion for the No. 8 orebody was determined (Table 7.5).

7.3 Numerical Modelling of the No. 8 Orebody

7.3.1 Problem Definition

The objective of the management at South Crofty is to optimise the efficient extraction of the ore reserves of the No. 8 orebody. At present the ore reserves of the No. 8 orebody lie between the 360 fathom level and the 420 fathom level (665 to 765 metres below the surface), although ore reserves in the No. 8 orebody are inferred down to 850 metres below the surface. The intention at South Crofty is to aim for 100% extraction, with no backfilling, but to leave pillars if it is deemed necessary. In order to determine whether and where pillars were required, a case study of the No. 8 orebody between the 360 fathom level and the 420 fathom level was conducted. The resulting stope design was intended to maximise the extraction of ore, whilst at the same time maintaining the stability of the surrounding rock mass. The open stopes created by ore extraction can be considered as temporary excavations but the haulage drive in the footwall of the No.8 orebody on the 400 fathom level is an important drive, both for tramming ore from the stopes, and for access to other parts of the mine. For this reason the No. 8 footwall drive on the 400 fathom level was required to remain open, and can be classed as a permanent excavation.

At the commencement of the author's research, stoping of the No.8 orebody between the 380 and 400 fathom levels had just begun. By May 1989 almost all of this ore had been removed, and the development of the orebody and footwall drives on the 420 fathom level was nearing completion (Figure 7.7). Thus, the

RMR' = 82	m = 13.1	s = 0.135
RMR' = 75	m = 10.2	s = 0.062
RMR' = 69	m = 8.3	s = 0.032

Table 7.5 The range of CSIR rock mass rating and corresponding m and s parameters, for the Hoek-Brown failure criterion, that are used in the numerical analyses.

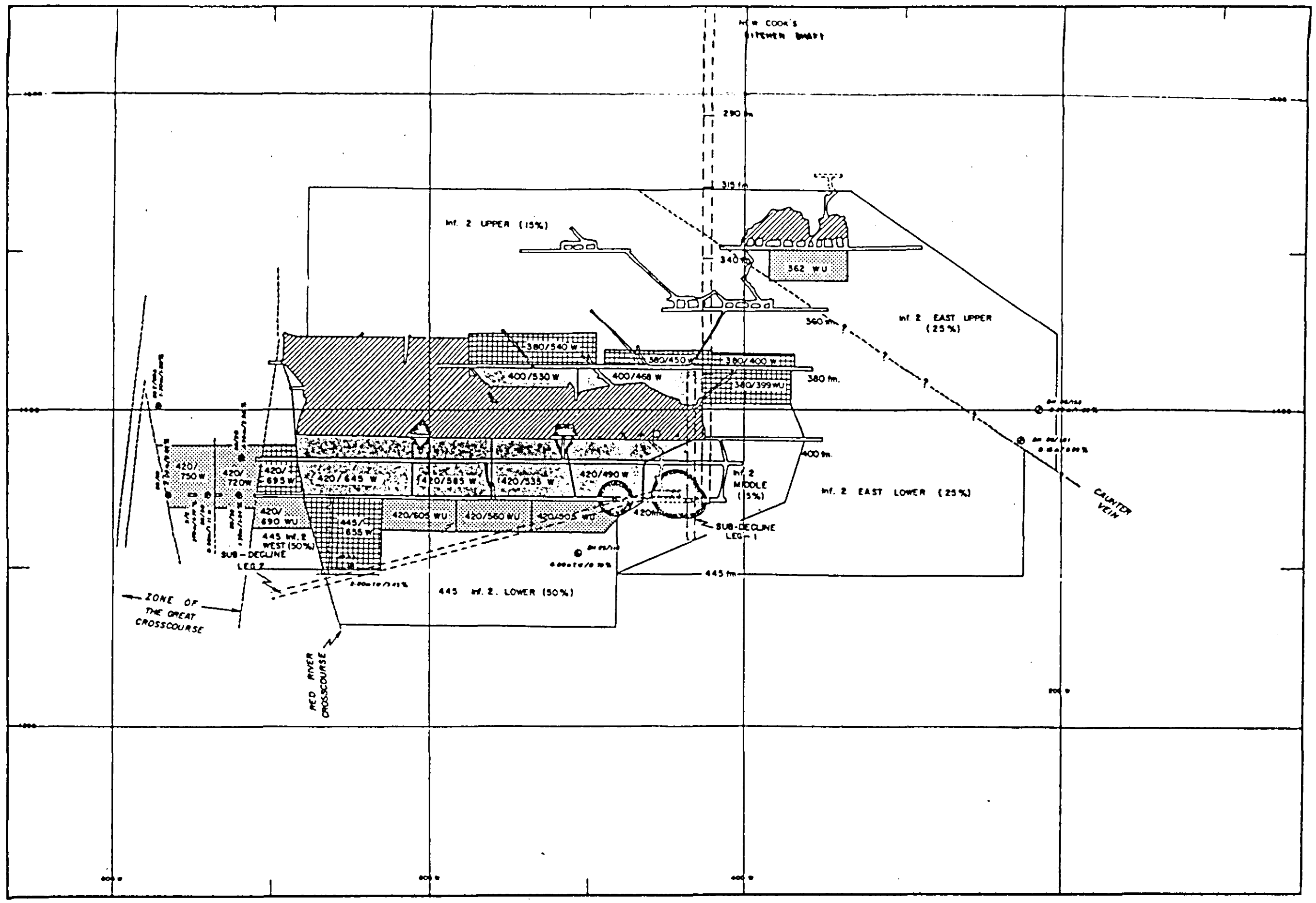


Figure 7.7 Longitudinal section of the No. 8 orebody.

mining plan to be devised for the No. 8 orebody would only affect stopes in this orebody between the 400 and 420 fathom levels, although the effect of these stopes and stopes on higher levels would have to be taken into consideration in any design. Several options for the mining plan became apparent, after discussions with South Crofty personnel, from a scheduling and ore grade viewpoint, and these are discussed below.

Option 1

The first option was to mine completely the area, from just above the 380 fathom level down to the 420 fathom level, and to leave no crown pillar below the 400 fathom level lode drive (Figure 7.8).

Option 2

The second option was to leave a substantial crown pillar below the 400 fathom lode drive (Figure 7.9). This crown pillar would have several roles. The crown pillar would be of a sufficient size, such that it would not fail, and would be able to transmit the high induced stresses. The crown pillar would also prevent waste rock, that might fall from the walls of the stopes above the 400 fathom level, going down into the stopes below the 400 fathom level, and hence leading to ore dilution.

Option 3

The third option was to leave a small crown pillar below the 400 fathom level lode drive, and a number of vertical rib pillars between the 400 fathom and 420 fathom levels (Figure 7.10). The role of the crown pillar would be merely to prevent the ore dilution that would occur if waste rock from higher stopes was allowed to fall freely down to the 420 fathom level. The crown pillar would be designed to fail non-violently, such that it would carry little stress, but would remain relatively intact. The rib pillars would be intended not to fail and to transmit the stresses induced by the removal of the supporting effect of ore.

7.3.2 Numerical Idealisation

The modelling aid part of the developed knowledge-based system for crown pillar design (Section 6.3) was used to conceptualise the behaviour of the rock mass and

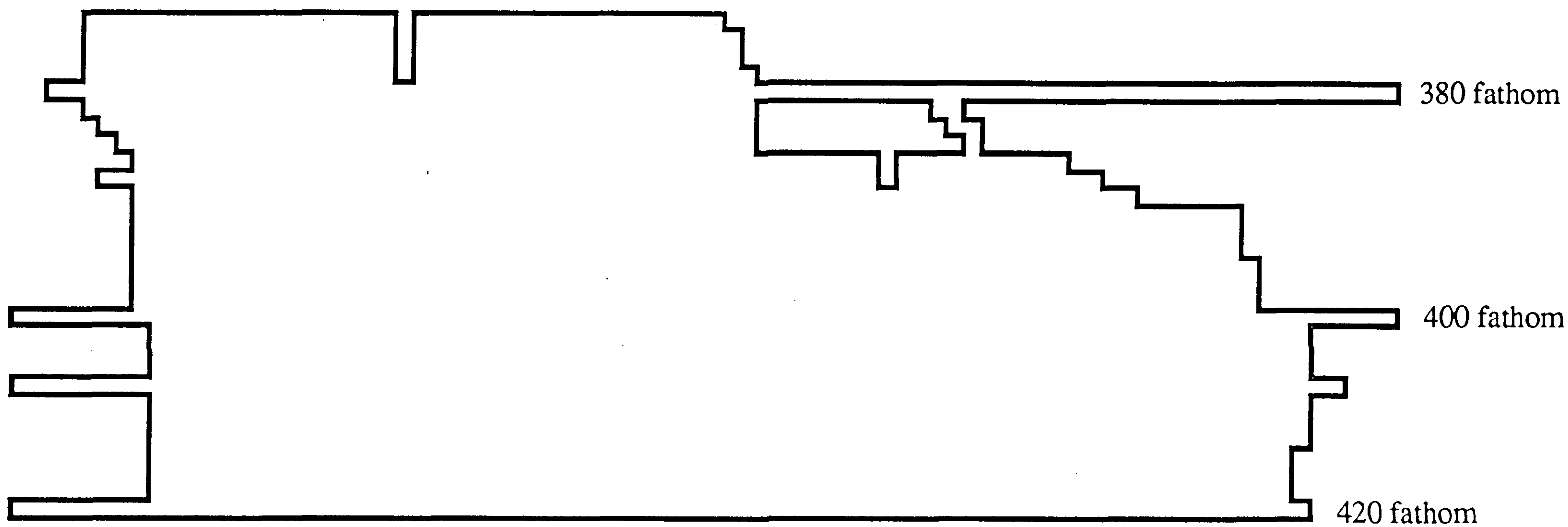


Figure 7.8 Schematic diagram of option 1: complete mining between above 380 fathom level and 400 fathom level.

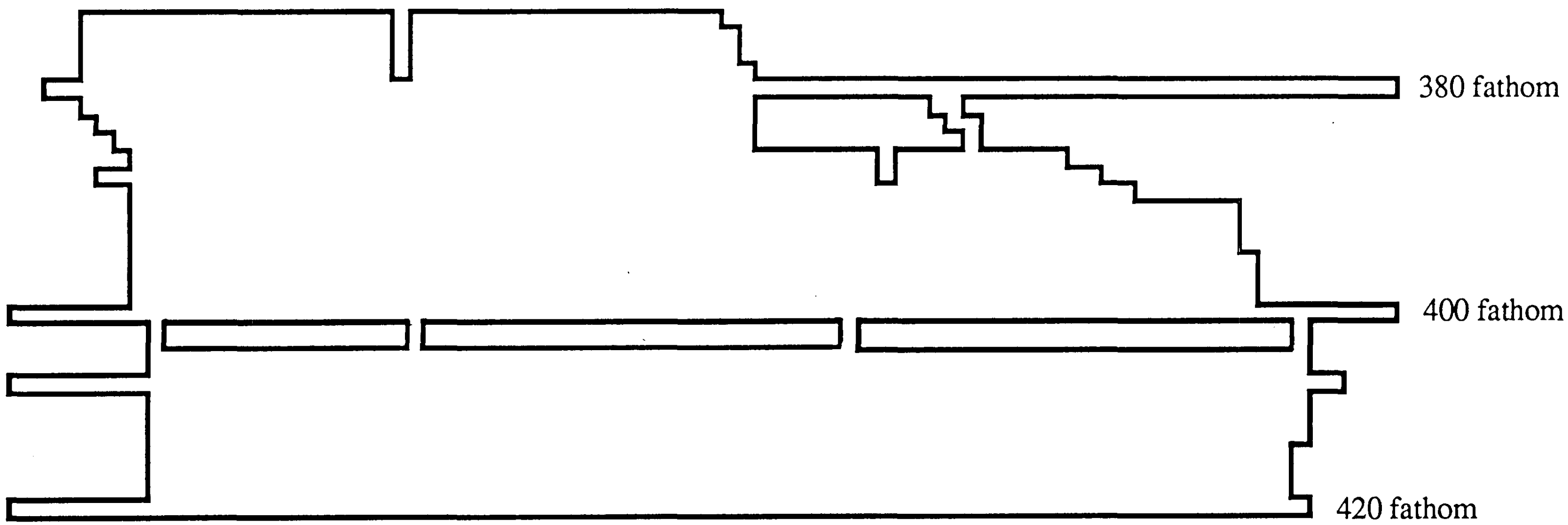


Figure 7.9 Schematic diagram of option 2: substantial crown pillar left below 400 fathom level.

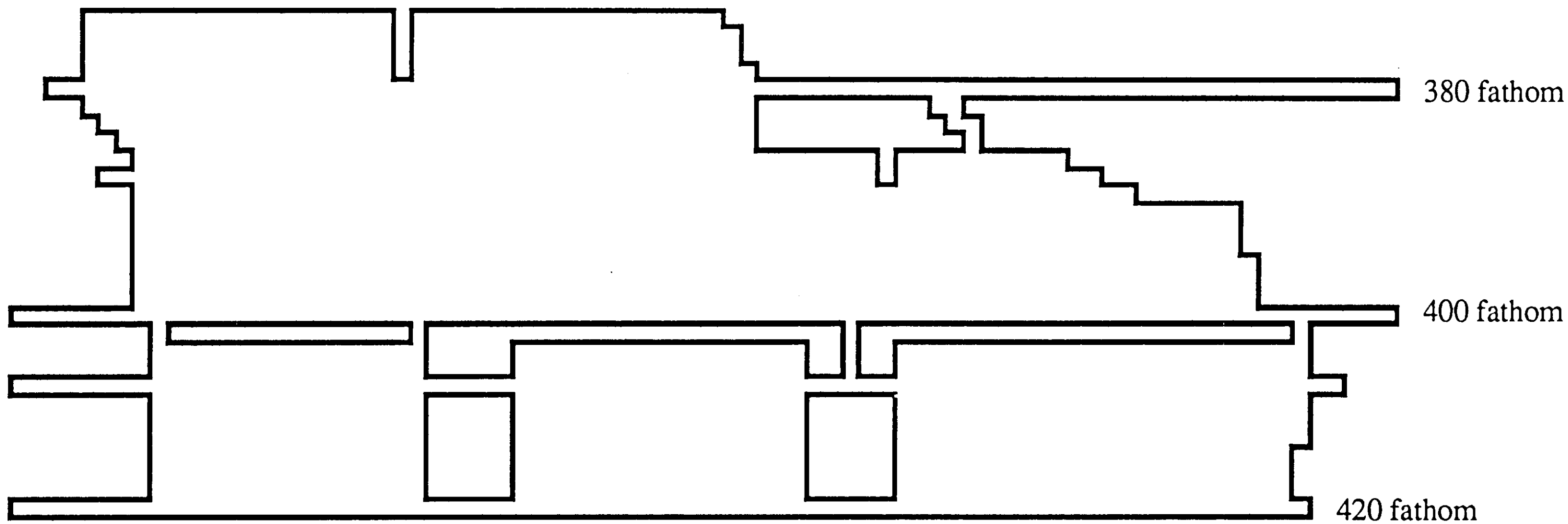


Figure 7.10 Schematic diagram of option 3: 3 m crown pillar below 400 fathom level and two rib pillars between 400 fathom and 420 fathom levels.

to determine which type of analysis was necessary for each of the three design options. The knowledge-based system concluded that the rock mass could be approximated as an elastic continuum and that linear elastic boundary element stress analysis was appropriate. A two-dimensional analysis was recommended for options 1 and 2, but a three-dimensional analysis was recommended for option 3. The reason for this was that plane strain conditions could be approximately assumed for a vertical cross-section through the middle of the No. 8 orebody, around 550W, in the case of options 1 and 2. For option 3, however, plane strain conditions would not exist in the rib pillars and so three-dimensional modelling of the stope and pillar layout would be required. In addition, the knowledge-based system concluded that conventional plane strain analysis was applicable for options 1 and 2 and suggested that the program TWOFS be used. For option 3, the system suggested using the three-dimensional displacement discontinuity program, MINTAB.

As part of the rock mass conceptualisation, the rock mass rating (RMR') and the rock quality index (Q') were calculated. These were found to agree closely with those given in Table 7.4 and so it was decided to model the No. 8 orebody using the range of Hoek-Brown failure criteria given in Table 7.5. The criterion for $RMR' = 82$ was viewed as being representative of the rock mass in general, but the value of RMR' is likely to vary along the strike length of the orebody.

7.4 Numerical Modelling Results

7.4.1 Modelling of Option 1

Option 1 was the complete excavation of all ore in the area, from just above the 380 fathom level down to the 420 fathom level, leaving no crown pillar below the 400 fathom level (Figure 7.8). This stope and pillar layout was analysed using TWOFS and the resulting tensor plot of the principal stresses σ_1 and σ_3, is given in Figure 7.11. The true dip length of the stope is around 120m and from Figure (7.11) it can be seen that a very large tensile region develops in the hangingwall and footwall of the stope, especially in the hangingwall above the 400 fathom level. This large size of this tensile zone is due to the change in dip of the orebody around the 400 fathom level and also to the above average width of the stope above the 400 fathom level. In the model the tensile zone extends up to 25 metres into the

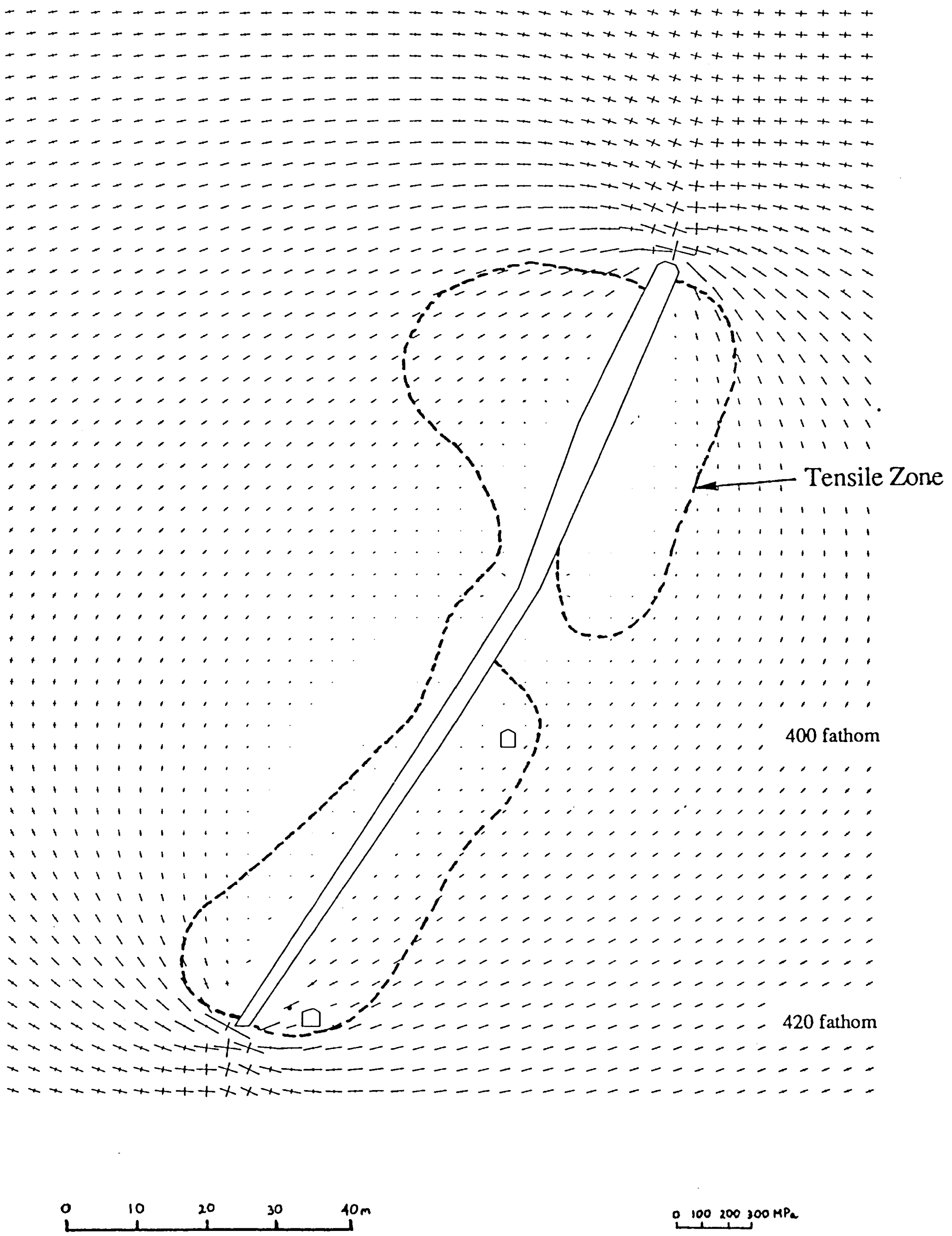


Figure 7.11 Tensor plot of principal stress distribution for option 1.

hangingwall and it is clear that this hangingwall will be unstable. The result would be considerable hangingwall collapse and dilution of ore.

Another consideration for the design was the stability of the 400 fathom level footwall drive, as this is a major haulageway and access route to other orebodies. For this reason it was required to remain stable, with only local failure being tolerated. Prior to mining below the 400 fathom level, the 400 fathom level footwall drive was in a compressive stress regime. After the stope is mined down to the 420 fathom level, the stresses would be redistributed and the 400 fathom footwall drive would be located in a zone of tension, in the stress shadow of the stope. This would lead to a change in failure mechanism in the footwall drive. Previously, only small amounts of stress spalling were evident and this was mainly due to the footwall drive having a square cross-section, with resulting stress concentrations in the drive corners. After mining below the 400 fathom level, the rock would be under a low compressive or tensile stress regime, which would allow movement of rock blocks by gravity. The footwall drive would require support, in the form of rock bolts, in both the walls and roof.

Option 1 would maximise the recovery of the ore extraction between the 400 and 420 fathom levels, but the subsequent increase in stope dilution and the necessary extra support that would be required for the 400 fathom level footwall drive, make the design option unattractive.

7.4.2 Modelling of Option 2

Option 2 was to leave a substantial crown pillar below the 400 fathom level, such that the pillar remained stable and transmitted the high induced stresses. This pillar would also prevent waste rock, from the hangingwall of the stope above the 400 fathom level, falling into the stope below. The design problem was to define the size that the pillar should be.

A series of stope layouts, each with a different pillar size, were modelled using the two-dimensional boundary element program TWOFS. For each stope layout the three different failure criterion given in Table 7.5 were used, to account for the variability in the quality of the rock mass along the length of the crown pillar. Figures 7.12 to 7.16 show tensor plots of the principal stresses σ_1 and σ_3 for each stope layout. It is clear that the tensile zone for the upper stope is relatively constant in size, irrespective of the size of the crown pillar between the upper and

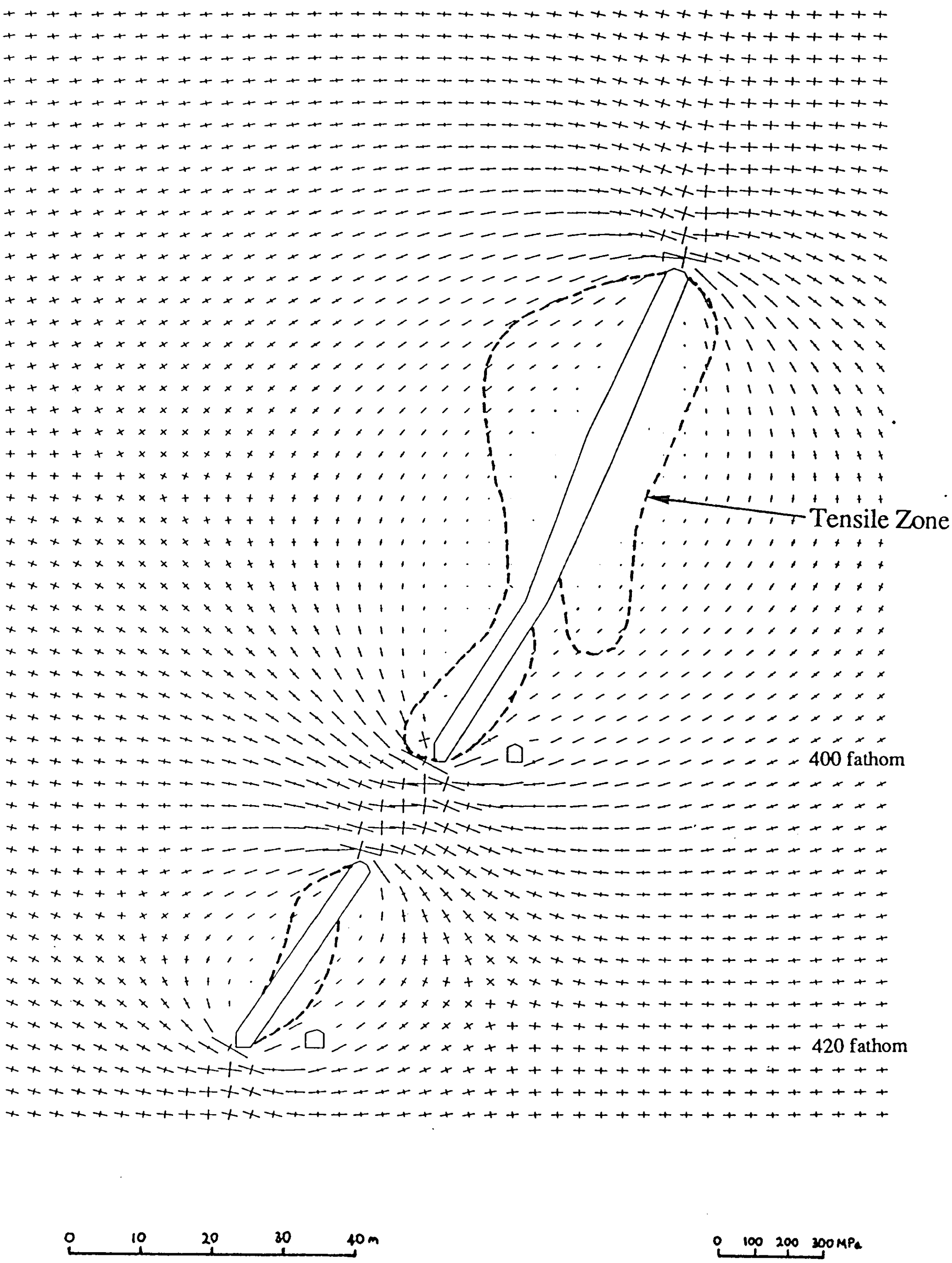


Figure 7.12 Tensor plot of principal stress distribution for option 2: 18 m crown pillar below 400 fathom level.

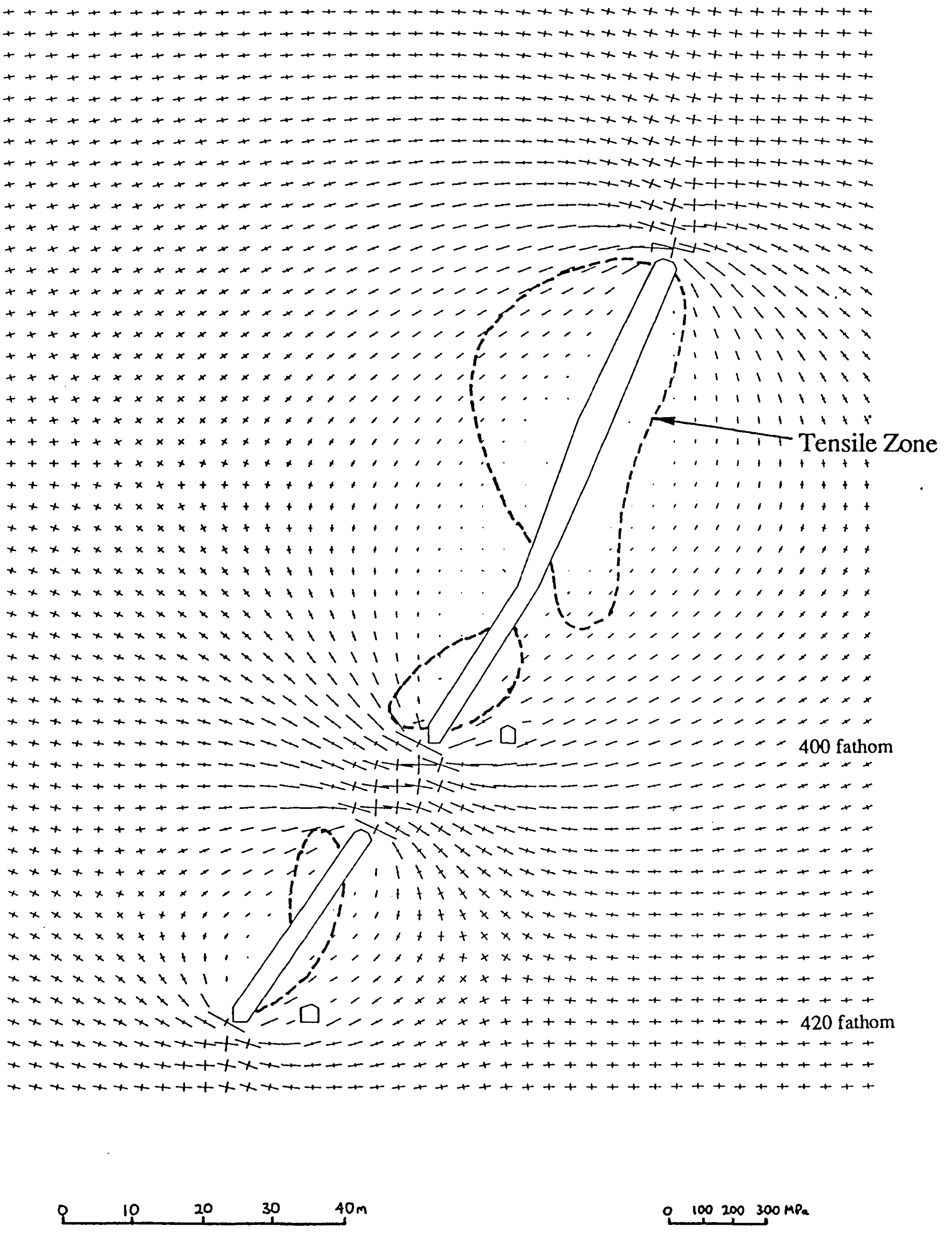


Figure 7.13 Tensor plot of principal stress distribution for option 2: 15 m crown pillar below 400 fathom level.

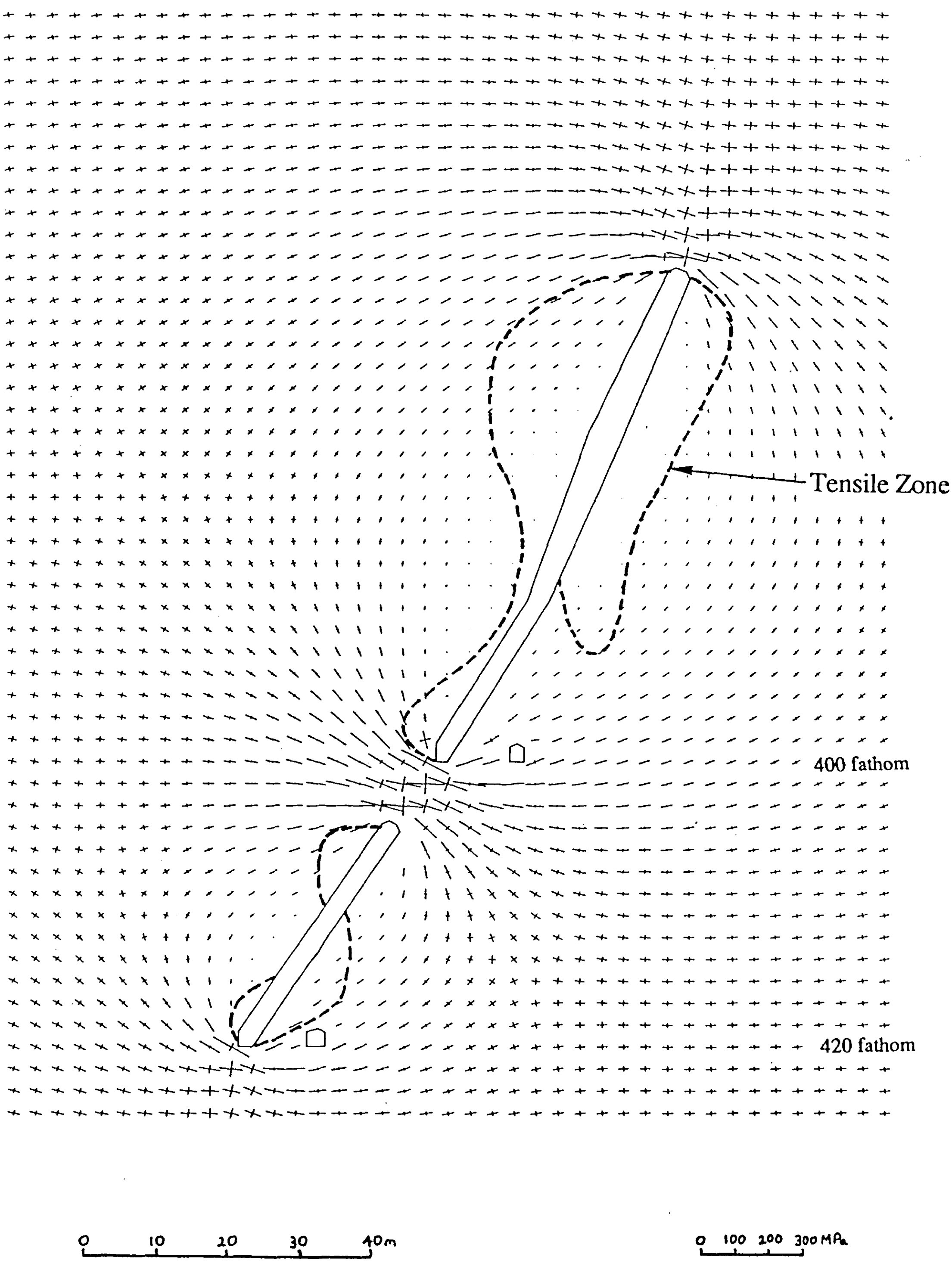


Figure 7.14 Tensor plot of principal stress distribution for option 2: 10 m crown pillar below 400 fathom level.

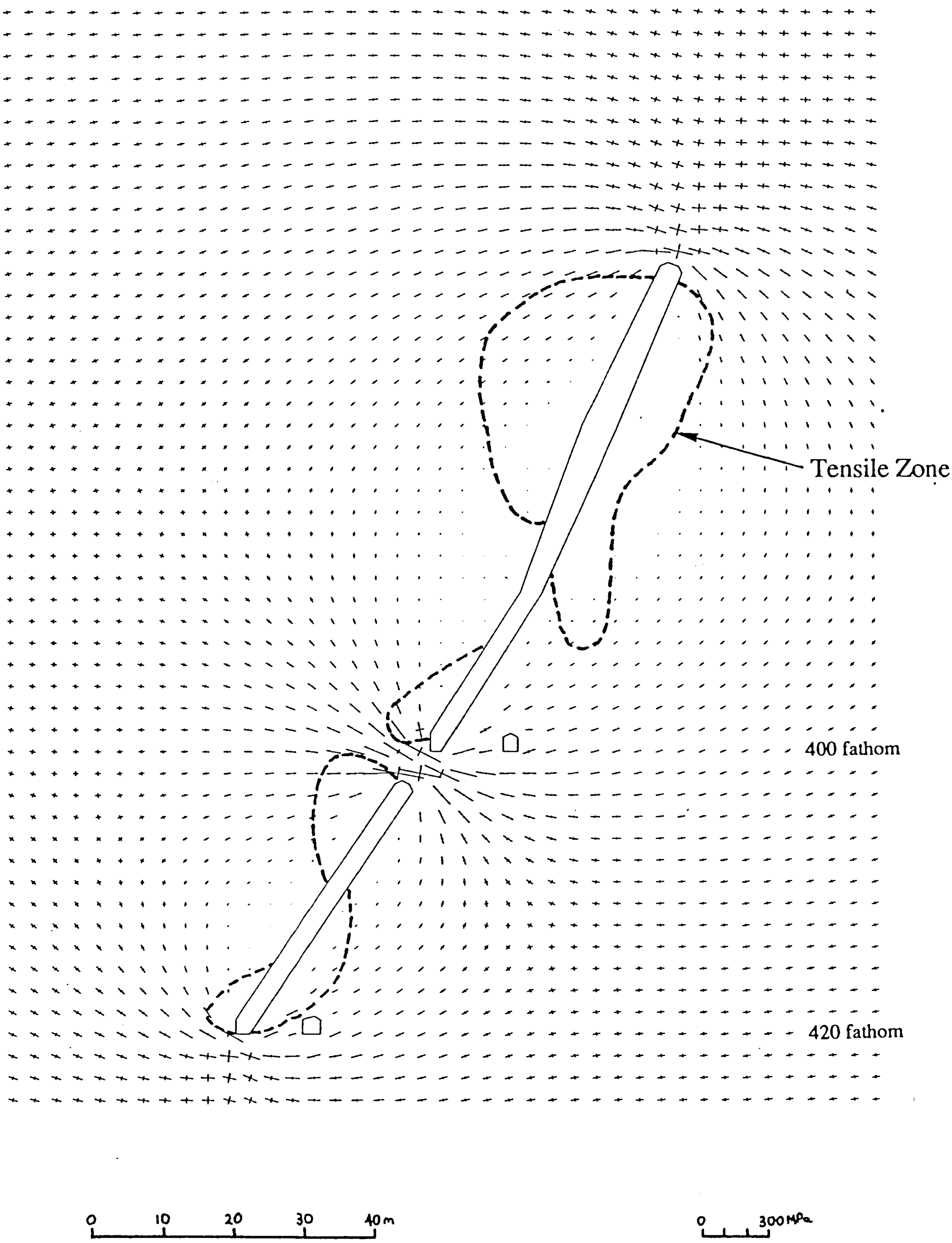


Figure 7.15 Tensor plot of principal stress distribution for option 2: 5 m crown pillar below 400 fathom level.

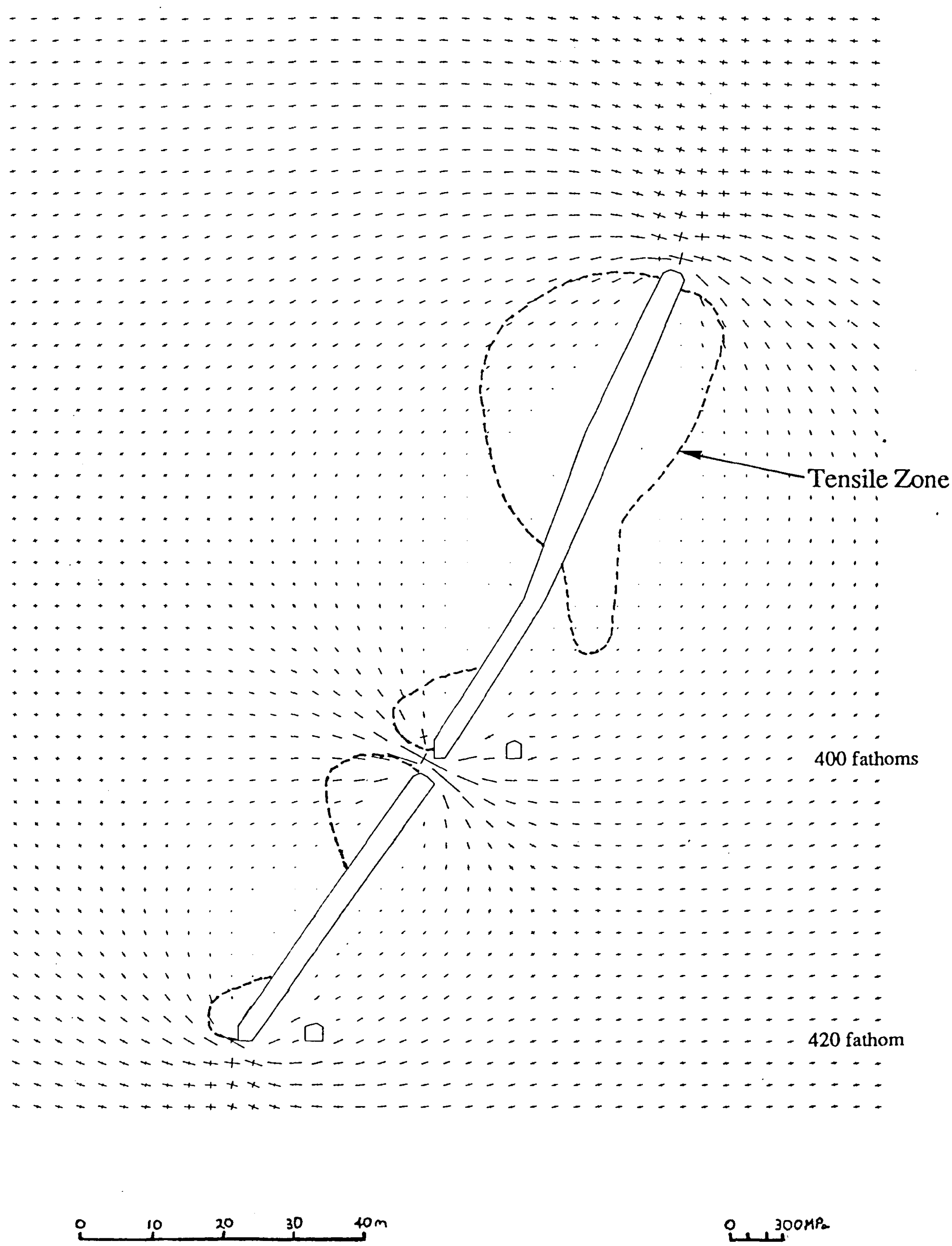


Figure 7.16 Tensor plot of principal stress distribution for option 2: 3 m crown pillar below 400 fathom level.

lower stopes, extending up to around 17 metres into the hangingwall. The tensile zone for the lower stope is affected by the crown pillar size, but it is quite small and generally localised. For the largest pillar modelled (Figure 7.12), the tensile zone extends about 4 metes into the hangingwall, whilst for the smallest pillar modelled (Figure 7.16), the tensile zone extends about 10 metres into the hangingwall.

In the crown pillar the stress distributions are very different, for the different pillar thicknesses. Using the Hoek-Brown failure criterion, the average factor of safety was calculated for each pillar, along a line from the roof of the lower stope to the floor of the upper stope. Figure 7.17 shows the variation of factor of safety with pillar thickness, and also shows the effect that the variation in RMR' has on the factor of safety.

The crown pillar for option 2 was required to remain stable. As mentioned in section 4.4.2, an average factor of safety below 1 is interpreted as failure, between 1 and 1.3 as temporarily stable, between 1.3 and 1.6 as stable and greater than 1.6 as suitable for permanent structures. Since the crown pillar is required to prevent waste rock from above the 400 fathom level falling onto and diluting ore in the stope below, then the crown pillar must remain relatively intact. The pillar must also be able to transmit the high induced stresses and not yield. For these reasons the crown pillar should be classed as a permanent structure and the average factor of the pillar should exceed 1.6. From Figure 7.17 it can be seen that an 8 metre crown pillar would have a factor of safety greater than 1.6 for the intermediate RMR' value ($RMR' = 75$) and has a factor of safety of around 1.4 for the lower band of RMR' ($RMR' = 69$). Thus, taking into account the natural variation in the rock mass condition along the crown pillar, an 8 metre pillar should be generally very stable and be large enough to perform the desired role.

As a check to the two-dimensional analysis, a three-dimensional analysis using MINTAB was conducted, for the extraction of the lower stope. Figure 7.18 shows the stress distribution normal to the orebody when only the upper stope has been mined and Figure 7.19 shows the stress distribution after the lower stope has been mined. It is clear from Figure 7.19, that plane strain conditions exist in the crown pillar at 550W (where the vertical cross-section for the two-dimensional analysis was taken), as the stress distribution is uniform along strike in the vicinity of 550W. It is also clear that the centre of the pillar is not very highly stressed, with the stress normal to the centre of the pillar predicted to be around 75 MPa.

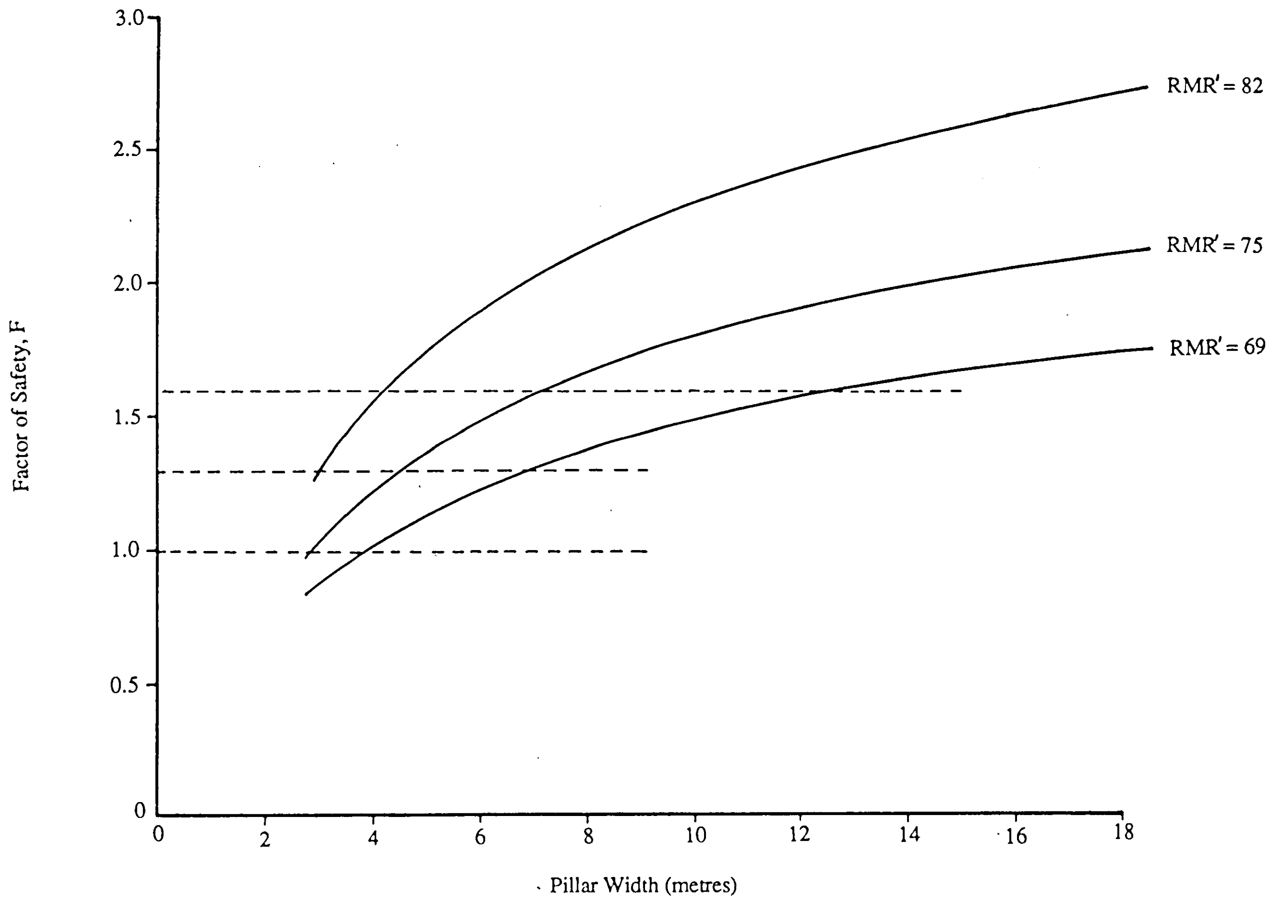


Figure 7.17 Variation of factor of safety with pillar width for the crown pillar below the 400 fathom level in the No. 8 orebody.

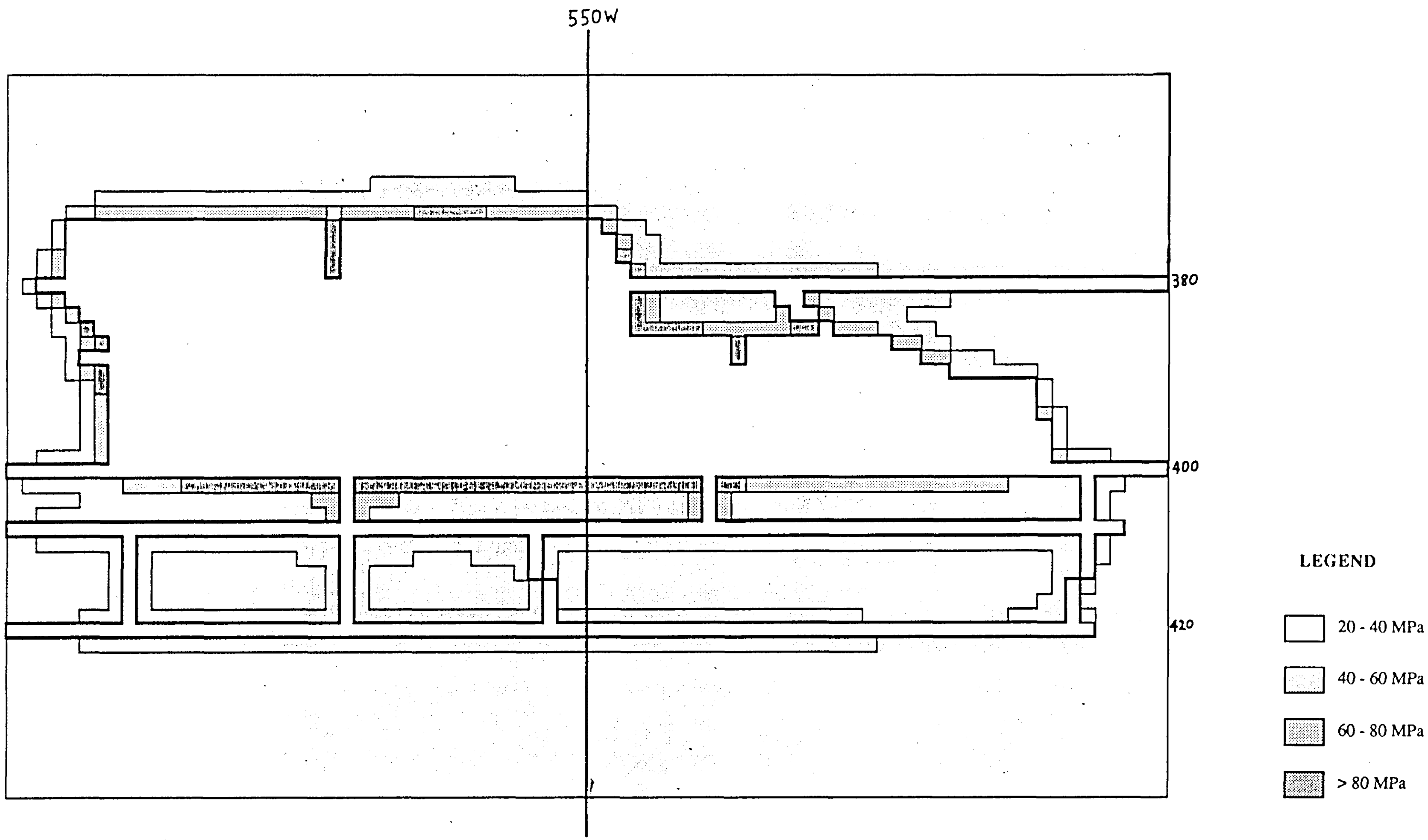


Figure 7.18 MINTAB modelling of option 2 – initial state.

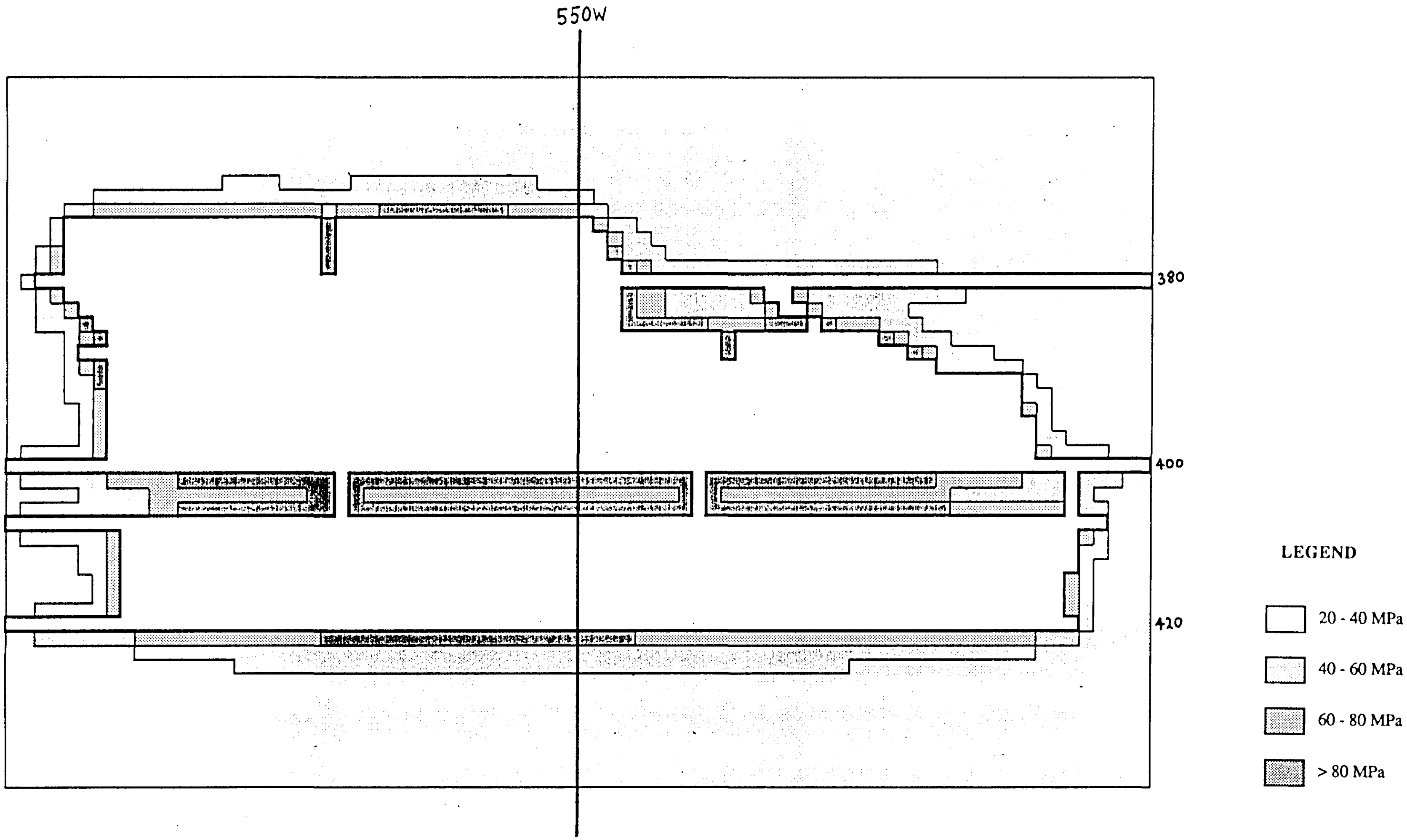


Figure 7.19 MINTAB modelling of option 2 – final state.

The edges of the pillar are more highly stressed and some local spalling would be expected, but the core of the pillar would remain stable.

The condition in the 400 fathom level footwall drive, as can be seen in Figures 7.12 to 7.16, would be affected by the size of the crown pillar. For an 8 metre crown pillar the footwall drive would be in a zone of medium compressive stress, as the high stresses tend to be transmitted below the footwall drive due to the dip of the stope and the position of the drive relative the floor of the upper stope. An increase in stress spalling would be expected and support, in the form of rock bolts, would be necessary.

7.4.3 Modelling of Option 3

Option 3 was to leave a small crown pillar below the 400 fathom level and two rib pillars between the 400 fathom and 420 fathom levels (Figure 7.10). The crown pillar was designed, in this case, merely to prevent dilution of ore in the lower stope from waste rock falling from the hangingwall of the upper stope. The role of the rib pillars, was to remain intact and to provide support for the hangingwalls of the two stopes.

The three-dimensional displacement discontinuity program MINTAB was used to model the progressive excavation of the lower stope. Four mining steps were modelled:

(1) Excavation of the upper stope (Figure 7.20)

(2) Excavation of the central portion of the lower stope (Figure 7.21)

(3) Excavation in the Western portion of the lower stope (Figure 7.22)

(4) Excavation in the Eastern part of the lower stope (Figure 7.23).

The rib pillars were each 20 metres wide and were designed to remain stable. As can be seen from Figures 7.20 to 7.23, the stress in the abutments of all the stopes increased as ore was removed. However, the unmined ore would not be so highly stressed as to make subsequent stoping too difficult, and the ore would be in a good condition when mined. The direction of mining within these blocks will also have an influence on the condition of unmined ore, and as a general rule ore should be mined in a direction towards the abutments. For the central portion of the lower stope, mining can be in either direction. For the Western portion of the lower

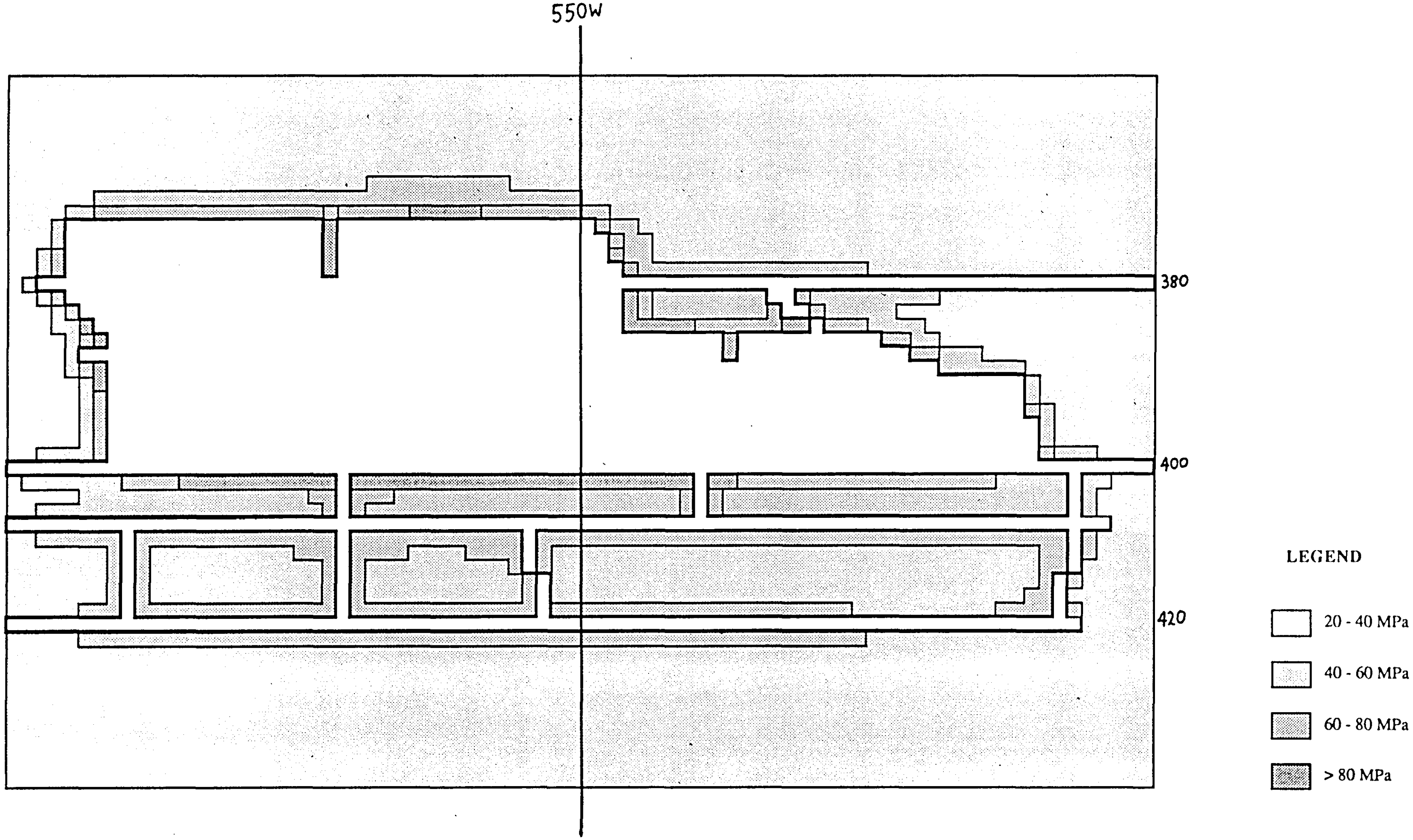


Figure 7.20 MINTAB modelling of option 3 – initial state.

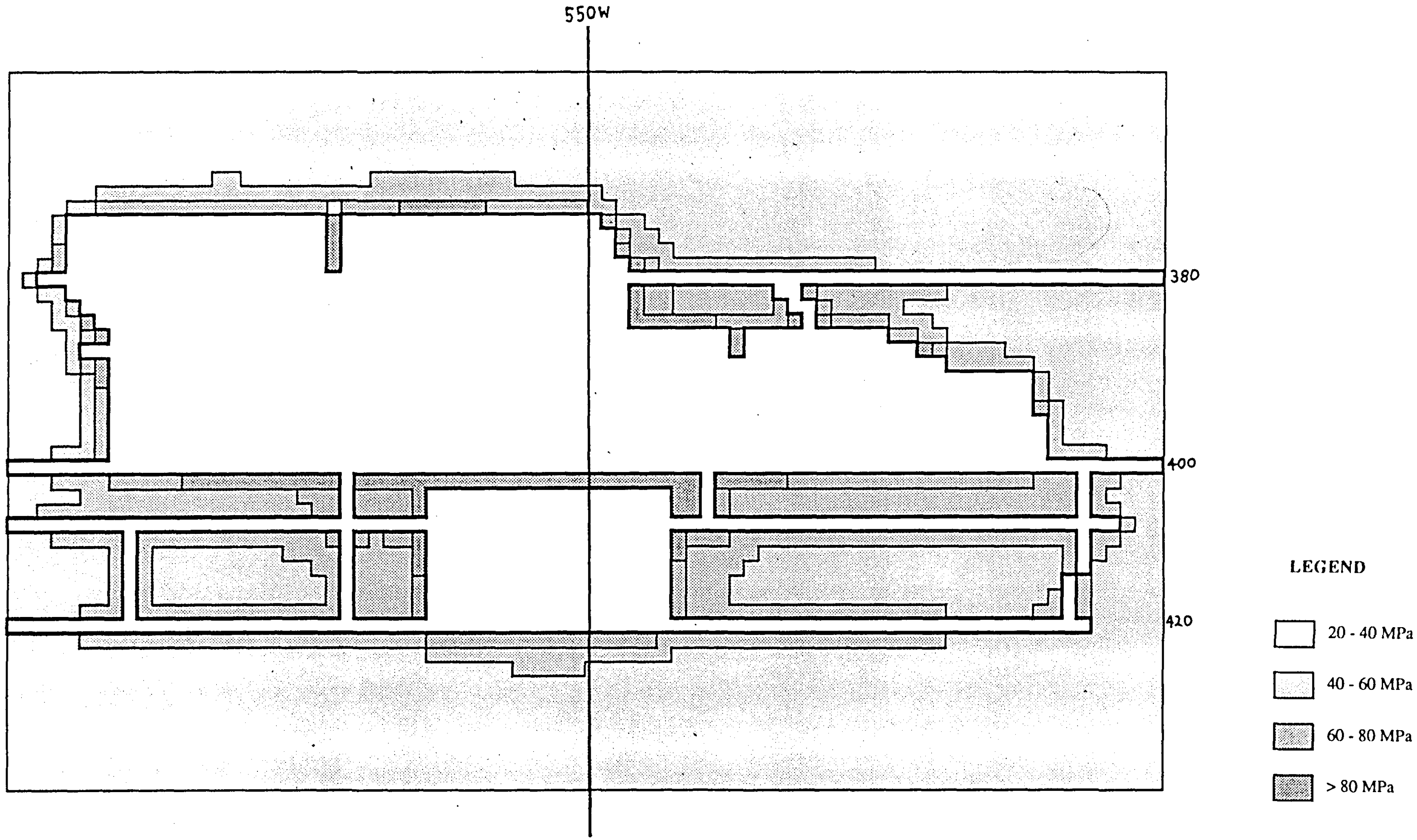


Figure 7.21 MINTAB modelling of option 3 – mining of lower central stope.

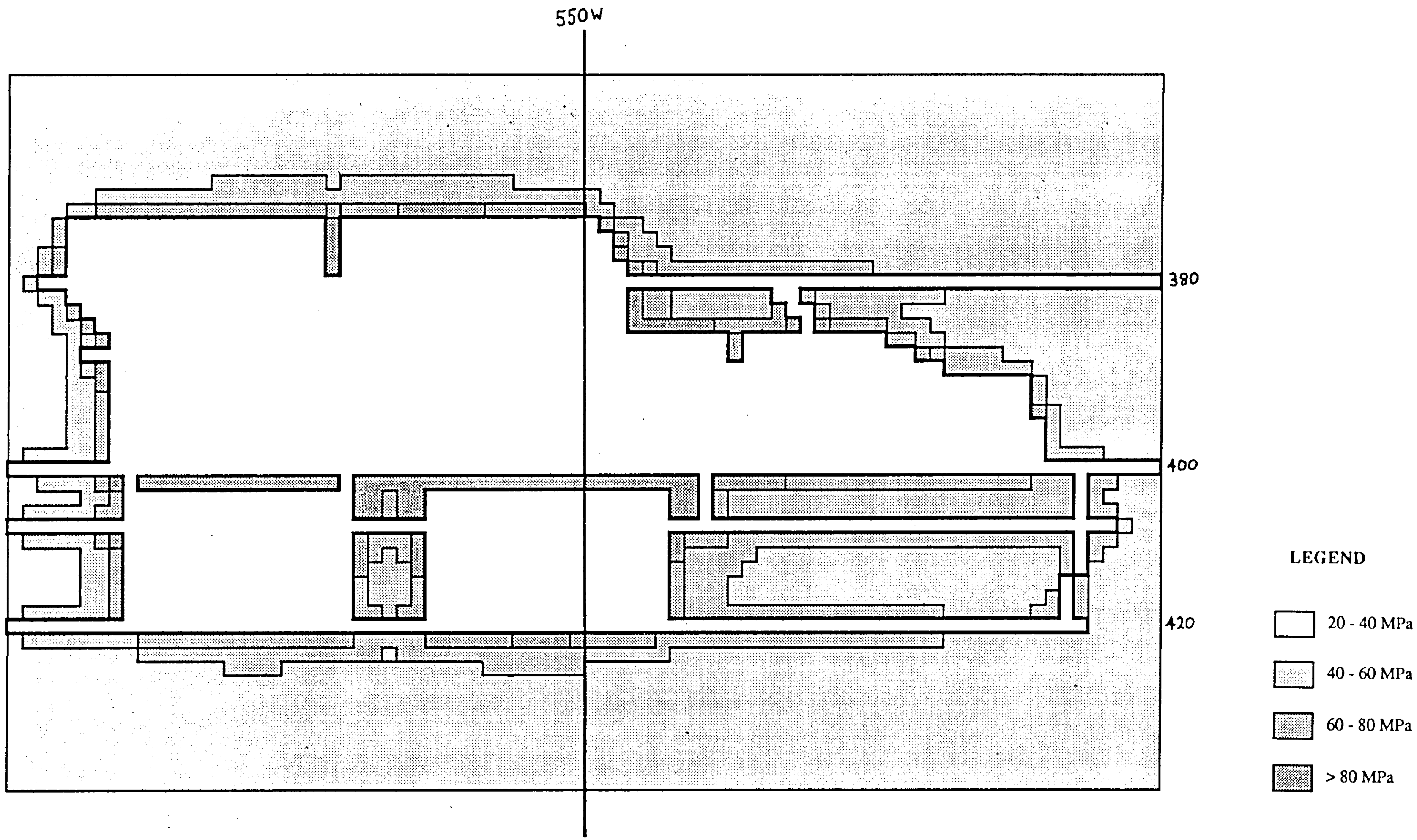


Figure 7.22 MINTAB modelling of option 3 – mining of lower Western stope:

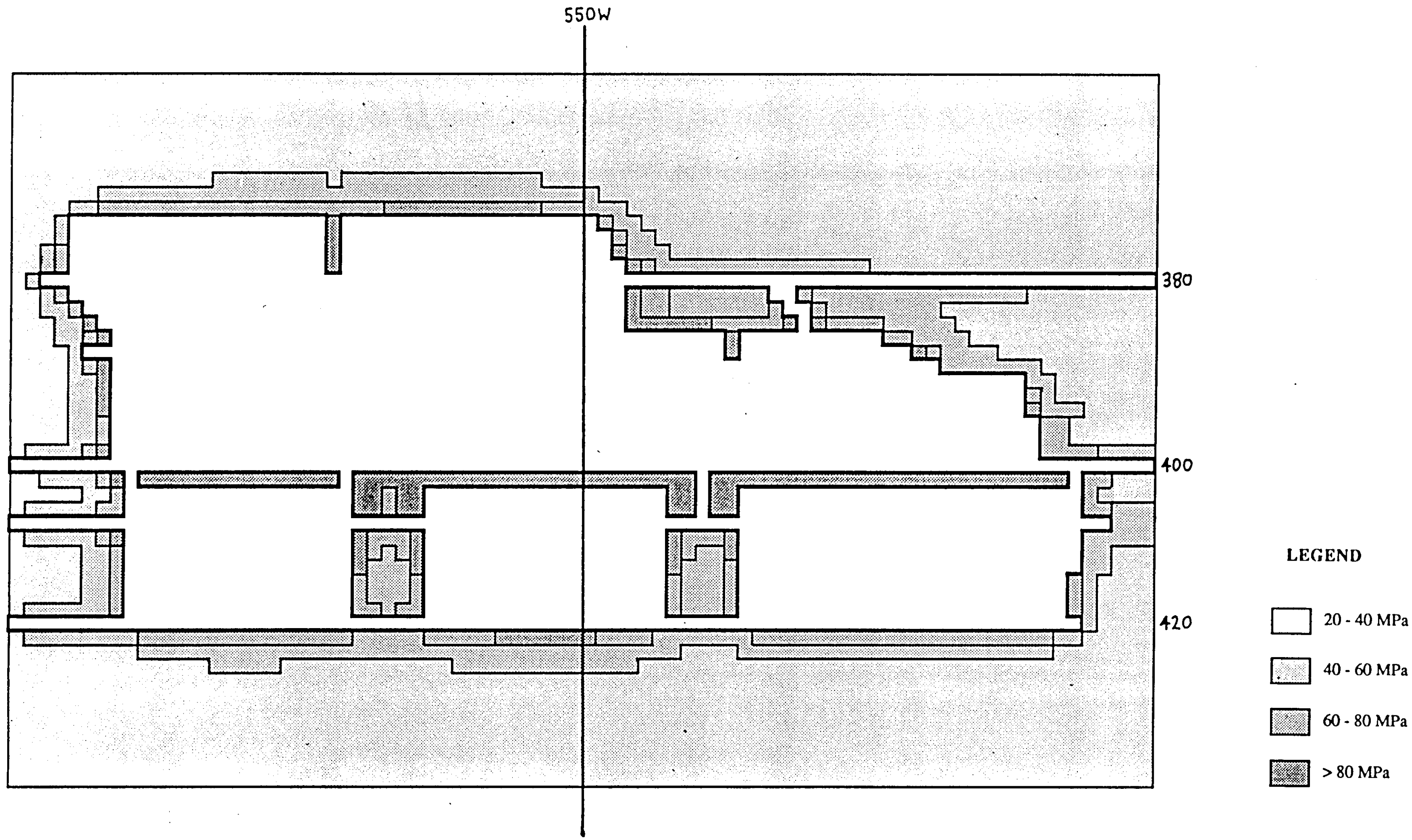


Figure 7.23 MINTAB modelling of option 3 – mining of lower Eastern stope.

stope, mining should progress from the rib pillar towards the Western abutment. For the Eastern portion of the lower stope mining, should progress from the rib pillar towards the Eastern abutment.

The main stress changes, due to mining, are noticed around the 420 fathom level lode drive and in the rib and crown pillars. In Figure 7.23 the final stope position is seen to result in a very variable stress distribution along the length of the 420 fathom level lode drive. This is due to rib pillars which attract stress and reduce stress levels in their vicinity. This variation in stress will have an effect on the condition of the 420 fathom level footwall drive. Spalling, due to over-stressing, will be worse in the footwall drive near the middle of the central stope (Figure 7.23).

The rib pillars, although highly stressed, should remain relatively intact, with the normal stress in the core around 70 MPa. At 20 metres wide these pillars are large enough such that they will remain relatively stable and amenable to later mining, while also having a positive influence on hangingwall stability. From the modelling, it is clear that the 3 metre wide crown pillar will become very highly stressed, such that it will begin to yield. The pillar, though, should remain relatively intact, but it would no longer be able to transmit stress. In this state, however, the pillar is still fulfilling its role, as a means of preventing waste rock from the upper stope from falling into the lower stope.

7.5 Conclusions and Practical Implications

The modelling aid part of the developed knowledge-based system for crown pillar design was of use in determining the most appropriate type of analysis for the three design options. For options 1 and 2, where two-dimensional analysis using the boundary element program TWOFS was recommended, then the intelligent front-end part of the developed knowledge-based system was used for determining the correct input parameters for the analysis and for writing the datafile for the numerical analysis.

The analysis was conducted using the developed open stope pillar design methodology, that was described in Section 4.4.2. The analysis represents only an initial analysis, as no monitoring of the No. 8 orebody was available in order to compare predicted behaviour with monitored behaviour. Visual observation of mining in the upper stope (between above the 380 fathom level to the 400 fathom level) compared well with the predicted behaviour of the hangingwall and the 400 fathom level footwall drive. However, the development of the 420 fathom stope is unknown territory for the mine, in that there are no comparable stope layouts in the No. 8 or any other orebody at South Crofty. There are no stopes that have a true hangingwall dip length of 120 metres as option 1 would produce and there are no large, fairly wide (+2 m), open stopes with large crown or rib pillars as options 2 and 3 would produce, respectively.

For the design loop in the developed methodology to be completed (Figure 4.13) stress and displacement monitoring of the crown and rib pillars are necessary, as are closure measurements in the 400 and 420 fathom level footwall drives. Using these monitoring data, the predicted and actual rock behaviour can be compared, to determine whether the failure mechanisms are as predicted, and to calibrate the numerical model to the site conditions, so that it may be used as a predictive tool for further excavations in the same orebody. For crown pillar design, the understanding of the pillar failure mechanism is of vital importance. If monitoring of the pillar shows that the pillar does not behave approximately as an elastic continuum but allows slip along planes to occur, then boundary element programs like TWOFS are inadequate. In such cases, it is important to model the failure mechanism using a program like MINAP (Section 2.3.5). If the pillar is shown to behave elastically, then monitoring of changes in stress and displacement can be used to modify the m and s parameters for the Hoek-Brown failure criterion, so that the failure criterion more accurately predicts the rock mass behaviour.

Using the developed open stope and pillar design methodologies, it is possible to optimise ore extraction. Future stopes at South Crofty should be designed using this methodology, with emphasis placed on good initial design. Initial design is very important in open stope mining, as the method precludes major changes to the design, once decided. One problem connected with this, is that little information is available for lower areas of an orebody, except for a few diamond drill cores. Thus, level and sublevels are usually at set depth intervals, and design is directed towards the layout of stopes and pillars. The developed stope and pillar methodologies can, however, accommodate changes in layout and hence, as more information becomes available about the ore to be extracted, the orebody model can be recalibrated.

The critical areas of the No. 8 orebody design are the 400 fathom and 420 fathom footwall drives' support, pillar stresses and hangingwall stability. Future stope and pillar design in the No. 8 orebody requires instrumentation and monitoring, to allow the comparison of predicted and actual stresses and displacements, so that the orebody model can be calibrated to site conditions. The following recommendations are made:

Footwall Drives

Convergence arrays should be installed in both the 400 fathom and 420 fathom footwall drives, preferably adjacent to the rib pillars and equidistant between them and the abutments. Between 5 and 7 arrays should be adequate. Measurements should be taken on a regular basis. If large movement is detected ($> 20\ mm$) then instability may be occurring and the support of the footwall drives should be further assessed. The rate of convergence as well as absolute measurements is also an important factor, as a rapid increase can indicate the onset of major instability.

Pillar Stresses

The optimisation of pillar dimensions, in advance of stoping taking place, is of great importance in open stope mining, as subsequent mining of pillars is often difficult and expensive. Initial under-dimensioning of pillars, however, can lead to bad ground conditions, making mining slow and difficult (and hence more expensive) and also increasing dilution, with the added risk of regional instability. The developed pillar methodology was shown to be of use in the optimisation of the thickness of the crown pillar below 400 fathoms in the No. 8 orebody. This was for an initial design layout, with the failure criterion for the rock mass derived from rock mass classifications of the orebody.

The use of CSIR stress cells would allow pillar design to be calibrated to site conditions, and enable TWOFS and MINTAB to predict induced stresses due to mining. Between 4 and 5 pairs of stress cells should be located within the crown pillar or the rib pillars, depending on whether design option 2 or 3 is selected. For option 2, where a substantial crown pillar is to be left, 4 stress cell pairs should be located at regular intervals along the strike length of the crown pillar. For option 3, where a small crown pillar and two large rib pillars are left, then two pairs of stress cells should be located in the bottom and middle of the rib pillars.

Once mining of the 400–420 fathom stope begins, stress changes can be compared to those predicted by the numerical model, in order to determine whether the general trends predicted by the numerical modelling match those measured in the rock. The stress data can also be used to calibrate the orebody model to site conditions, by modifying the parameters for the failure criterion, so that the calibrated model can be used as a predictive tool.

Hangingwall Stability

For future stope layouts, the size of stope hangingwalls is an area that needs careful consideration, especially in the initial layout. In the No. 8 orebody, above the 400 fathom level, the hangingwall is very extensive along strike and a well developed tensile zone exists which has resulted in problems with dilution. The stability of the hangingwall cannot be measured directly using extensometers, as there is no access for their installation. Special hangingwall drives, to allow installation of these extensometers down into the hangingwall, would be prohibitively expensive.

At present, there is no systematic recording of dilution levels, merely general feelings about dilution levels. Dilution is a good measure of the success of an open stope design, providing that blasting is of sufficient quality. Where blasting is poor then the effect of stope design on dilution is masked by the blast induced dilution. Regular visual observation of dilution for each drawpoint, combined with regular assays of drawpoint grab samples, provides a reliable measure of the degree of dilution observed. These dilution values can then be linked with the Stability Graph method (Section 3.4.1), as a means of attaching quantitative measures to the qualitative descriptions of stope plane stability. If enough data from an orebody can be recorded, then the stability graph can be calibrated. This calibrated stability graph can then be used as a means of determining optimum stope hangingwall areas.

CHAPTER 8
SUMMARY AND CONCLUSIONS

A Brief Summary

Numerical modelling is now firmly established in engineering rock mechanics, as a means of determining the stresses and displacements around excavations. The available numerical methods were critically assessed in terms of their applicability to open stope and pillar design in hard rock mines, concentrating specifically on design in narrow, steeply dipping, tabular orebodies. The boundary element method was shown to have clear advantages over the other methods and a suite of boundary element programs were adapted for use in this research. Each program was then assessed in order to determine under which circumstances each could be applied, and what their individual strengths and weaknesses were.

The practical design of open stopes was investigated and three modes of design were identified. A modelling methodology using simple numerical programs, combined with an observational approach was selected as the most appropriate for open stope design. The issue of modelling philosophy was addressed and the use of conceptual models as a means of unambiguously describing a rock mass was proposed. Finally, a new advanced design methodology for open stope design was developed.

In order to understand pillar performance, the mechanisms involved in pillar failure were investigated. A review of open stope pillar design demonstrated that historically, pillar design had been by precedent practice, with little regard as to the mechanics involved. Numerical modelling of pillars was identified as an effective means of optimising pillar dimensions. An advanced design methodology that was based on numerical modelling, coupled with an observational approach, was developed.

The new technology of knowledge-based systems was discussed in detail, and the various application modes were described. The development of intelligent modelling aids and intelligent front-ends to algorithmic programs were identified to be areas where knowledge-based systems could have an important impact on engineering rock mechanics design. Utilising the design methodology developed for pillar design, together with compiled knowledge on numerical modelling, a prototype knowledge-based system for crown pillar design was developed and its use was described.

The practical application of the research was demonstrated by analysing an open stope layout in the No. 8 orebody at South Crofty mine. The developed methodologies for open stopes and pillars, together with the knowledge-based system, were used to optimise the dimensions of a crown pillar.

Main Conclusions

As conclusions were given in each chapter, only the main conclusions of the research are presented below.

(a) Numerical modelling has been used with some success in all areas of stope design but in many cases the full potential of numerical analysis has not been realised. This is because all numerical models have specific strengths and weaknesses and these have to be fully understood if the numerical model is to be correctly applied. The design problem that this research is concerned with is that of stope and pillar design in hard rock mines, and specifically design in narrow, steeply dipping, tabular orebodies, which are a common and important mineral source. Such orebodies can often be considered as behaving as an elastic continuum and simple elastic analysis is often acceptable. The boundary element method is shown to have clear advantages over other numerical methods for such analysis, in its ability to model infinite problems, in its small data set required, and in its computational simplicity. However, this does not mean that the boundary element method is always the correct method. The features of the rock mass and the design problem itself, determine which is the most appropriate numerical method.

(b) Modelling methodologies for rock mechanics have developed out of methodologies developed in other branches of engineering. However, a rock mass is not directly analogous to other engineering materials, whose properties are generally well understood. Rock is a natural material and its properties have to be measured rather than specified, as in many other branches of engineering. Conceptual models provide a framework for the unambiguous description of a rock mass. Conceptual models also force modellers to associate *in situ* conditions with those of the geological model, and emphasise the mechanics of failure. Using a conceptual modelling approach, field data can be used to select the appropriate conceptual model. Then, analysis and design work can take place within the context of the selected model, and design alternatives appropriate to the model can be identified.

(c) Open stope and pillar design has in the past relied heavily on past experience. If numerical modelling is to be used effectively in practice, then it must be part of a methodology developed specifically for open stope design. Simple conceptual modelling uses relatively simple numerical techniques, but its strength comes from the fact that it is designed specifically for rock mechanics design, and recognises the basic problems that arise in open stope design. These problems include the difficulty in determining the material properties and boundary loading conditions for the numerical analysis, together with the quality and quantity of such data, which are often low.

(d) The design of hangingwall dimensions is one area where numerical analysis has had little success. The lack of access to hangingwalls has prevented the failure mechanisms from being fully investigated, and incorporated into numerical programs. The stability graph method is an empirical design method that takes account of the main rock mass properties, together with the stress conditions present and the relative dimensions of the excavation face. The stability graph method offers mining engineers a tool for assessing hangingwall stability, and for predicting the maximum safe span. The incorporation of the effect due to cable bolting of stope faces, allows the mine operator to determine the necessary bolting required to stabilise the face. When case histories of stope dilution are linked to the design methodology, then the expected dilution for any stope can be predicted in advance. The predicted level of dilution for various densities of cable bolting can be determined, and the economics of stope support can be optimised.

(e) The nature of the open stoping mining method is such that once pillar dimensions have been selected, then it is relatively difficult and expensive to change the design. Under-dimensioning of pillars can lead to pillar failure and excessive dilution, as well as endangering regional stability. Over-dimensioning of pillars results in a loss of ore reserves, with the subsequent added expense if the pillar is later mined. Thus, crown pillar optimisation is seen to be a major concern in the mining industry. Pillar failure mechanisms are understood but pillar design is still predominantly conducted using empirical techniques, which take no account of the mechanisms involved.

The methodology that was developed for pillar design is similar to that developed for open stope design, in that a conceptual model of the rock mass is first developed that incorporates the important features of the rock mass. Using the conceptual model the most appropriate numerical analysis program can be selected. Back analysis of actual conditions to those predicted by the model, and the varying of input parameters until actual and model conditions are similar, results in a calibrated model, that can be used as a predictive tool for pillar dimensioning and pillar layout design. This observational approach is very important because of the complex behaviour of pillars and the inability to accurately model the stress distributions within them.

(f) Numerical modelling has not been used routinely in mine design but it is increasingly being incorporated into mine design methodologies. There are many reasons why numerical modelling has not been more widely used, but the main problems have been that there were no formalised design methodologies and that the numerical programs require skill to use effectively. The second of these problems was identified as being especially important if numerical analysis was to be used as a part of practical stope and pillar design methodologies. In particular, problems were identified in the selection of the most appropriate numerical analysis program, in the determination of the input parameters to a program, and in the transformation of the actual design problem into a form that could be understood by a numerical program.

The task of modelling an open stope and pillar layout for a mine, like many problems in engineering rock mechanics, requires judgement by a modeller, based on accumulated experience, that is largely heuristic in nature. Mine planning engineers are commonly neither expert modellers nor expert users of numerical analysis programs. Traditional formalised methods and algorithmic programming techniques have not been successful in encoding numerical analysis design knowledge in a way that allows non-experts to undertake correct and effective numerical modelling. Knowledge-based systems provide a framework whereby expertise and state-of-the-art techniques can be incorporated into a single system.

(g) Due to the large number of design problems that exist in open stope and pillar layout, attention was focused on a single issue, that of crown pillar design. This is perceived by many mines as being one of the most important design tasks. A knowledge-based system approach was thus adopted in order to encode the crown pillar design and analysis knowledge.

As the aim was only to demonstrate the applicability of knowledge-based system techniques to open stope and pillar design, the system needed to be rapidly prototyped. An expert system development environment (shell) was chosen as the most appropriate development tool because all the elements of a knowledge-based system were provided, and only the knowledge had to be coded in. This reduced the amount of time spent actually programming, and allowed more time to be spent conceptualising the problem, and formalising the knowledge structure.

The knowledge-based system application modes of intelligent modelling tools and intelligent front-ends to numerical programs were identified as being the most suitable for the specific problem. Rock mass classification was identified as being ideally suited to being incorporated in the knowledge-based system, as it involves a blend of geological knowledge, intact rock data and field data, much of which are highly subjective in nature. Using elements of rock mass classification, together with data on geometry, conceptual models of crown pillar behaviour were developed, that formed the core of the intelligent modelling tool. When the intelligent modelling tool was combined with knowledge about numerical modelling techniques and their applications, then the most appropriate numerical program, for the particular design problem and rock mass condition, could be selected.

Intelligent front-ends to the two-dimensional boundary element programs were developed, which led the user through the input of the required data for the numerical programs, and which then wrote the actual datafile for the program. The intelligent modelling tool and the intelligent front-ends together constituted the complete knowledge-based system for rock mechanics modelling of crown pillars in narrow, steeply dipping, tabular orebodies. However, the structure of the developed knowledge-based system was such that it could be expanded to incorporate other open stope mine design issues.

(h) The developed methodologies for open stope and pillar design were used in the case study, as was the knowledge-based system for crown pillar design. The case study demonstrated the ease with which such design analysis can be conducted in practice. If knowledge-based systems are to become accepted by the mining industry, however, they will have to be perceived as reliable and accurate. The only way for this to occur is if sufficient case histories and expert knowledge are incorporated, such that the systems are able to deal effectively with most situations.

Suggestions For Further Research

The developed knowledge-based system demonstrated the use and applicability of knowledge-based system techniques in open stope mine design, and also indicated that such techniques could be applied in all areas of engineering rock mechanics design, as a means of encoding knowledge in a more natural way than that allowed by conventional programming techniques. Limitations of the developed knowledge-based system are discussed in previous chapters, and further investigations are required. The following suggestions are made to improve the accuracy, reliability, and generality of the program.

(1) In open stope design, further work is required to understand the mechanisms of failure around stopes, especially hangingwall failure. Visual inspection of hangingwalls using borehole television cameras is one way of determining the extent of stress relaxation in hangingwalls, as well as assessing the effect joint orientation, joint spacing, and joint condition have on the failure process. The interaction between stope support, particularly cable bolting, and a rock mass also needs to be more fully understood, if stope support is to be more effective, and if it is to be more correctly incorporated into numerical models.

(2) In pillar design, further research is required to understand the mechanics of failure, especially the post-peak performance of hard rock mine pillars, where little research has been conducted to date. Full scale mine pillars in hard rock should be instrumented and adjacent excavations progressively mined

to induce pillar failure. The results from such tests could then be used to determine the appropriate analysis techniques, and to identify the important features of the pillar rock that ought to be considered in any model. The use of hybrid numerical programs in pillar design should be investigated, to see whether they can efficiently model the complex rock behaviour in a pillar, within a rock mass that behaves essentially as an elastic continuum.

(3) Design methodology will become an important area of engineering rock mechanics research, and knowledge-based system techniques provide a means for expressing and implementing methodology. In rock mechanics design work, the conflict between different objectives needs to be investigated, and the logical methodology that rock mechanics engineers use must be further studied. Open stope and pillar design is such a large subject area that no one person can be an expert in all aspects. Interviews with recognised experts in rock mechanics numerical modelling, stope and pillar design, and mine planners will allow their experiential and heuristic knowledge, as well as their technical expertise to be recorded. Using this knowledge it should be possible to record the logical inter-connections and resolutions between conflicting design issues.

(4) The developed prototype knowledge-based system for crown pillar design utilises simple concepts, and requires further development if it is to be used in practice. Future work should be directed towards enlarging the knowledge base to incorporate subdivisions of the developed conceptual models of rock masses, so that the rock behaviour can be defined more precisely. Linked to these new conceptual models could be a greater diversity of numerical programs, each with its own intelligent front-end. Programs that permit the modelling of non-homogeneity, anisotropy and joints could be included. In addition, work could be directed towards automating much of the input to the intelligent front-ends, especially with regard to stope discretisation and internal point location.

If the system were to be enlarged to incorporate a wider range of design issues (*e.g.*, stope dimensioning, stope layout, stope sequencing), then the rule-based representation scheme adopted in this research would become too restrictive. The use of a frame-based representation scheme would allow the complex interactions and associations in the design knowledge to be represented. If the system were then to be incorporated into a general mine design knowledge-based system, then a blackboard approach could be adopted for the communication between individual system modules. The advantage of such an approach would be that once the blackboard had been specified, then the individual design modules, that make up the total system, could be developed independently.

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GLOSSARY

Below is a glossary of terms used in the knowledge-based system literature.

Artificial Intelligence (AI)

The sub-field of computer science concerned with understanding human reasoning and thought processes, and the application of this understanding to the development of computer technologies. Branches of this discipline include (amongst others) robotics, vision, speech recognition and knowledge-based systems.

Backward Chaining (consequent or goal driven)

A reasoning strategy which uses the conclusions (THEN parts) of rules to guide the search for rules to execute. The rules that can satisfy a particular goal are collected and the IF parts of these rules are examined. If all the IF parts cannot be satisfied then a search is made for a rule that has any of these IF parts as a conclusion. Thus a tree-like rule structure is developed until the original goal can be satisfied.

Blackboard Model

The blackboard model was developed to provide a mechanism for reasoning about problems with multiple sources, or levels, of knowledge. In the blackboard model the knowledge base is separated into knowledge sources, and the blackboard is the place where messages between knowledge sources, as well as the current state of the problem, reside.

Domain Knowledge

The term domain refers to the specific area of application, such as geology, mining, rock mechanics. Domain knowledge is that knowledge which is specific to the domain.

Expert Systems (ES)

Computer programs that perform specialised tasks requiring knowledge, experience, or expertise in some field. Other terms for expert systems are knowledge-based expert systems (KBES) and intelligent knowledge-based systems (IKBS). A more descriptive term is knowledge-based systems (KBS).

Expert System Shell

A program that contains two of the three components that comprise a knowledge-based system – the working memory and reasoning strategy (also called the inference engine) – but lacks knowledge. In addition expert system shells usually contain an editor, that allows the knowledge to be coded, together with development aids such as debugging tools, user interface development facilities and explanation system tools.

Forward Chaining (antecedent or data driven)

A reasoning strategy in which the IF parts of all the rules are checked against the database of known facts and conclusions, to determine all the rules that have all their IF parts satisfied. If there is more than one rule that has all its IF parts satisfied, then one rule is selected by a conflict resolution strategy (usually the order of the rules). All the actions associated with that rule are then performed and the database updated. The same process is then repeated.

Frames

A knowledge representation scheme in which a collection of slots describe aspects of an object. These slots may contain values, default values, or pointers to other frames describing other objects.

Heuristics

Expert 'rules-of-thumb' that are usually empirical in nature, based on experience and intuition, with no mathematical or scientific proof.

Inference Engine

The software which controls the reasoning operations of a knowledge-based system. This is the part of the program that deals with making assertions, hypotheses and conclusions. It is through the inference mechanism that the reasoning strategy (or method of solution) is controlled.

Inheritance

A property of the frame representation scheme, whereby attributes from other frames are said to be inherited via pointers that relate one attribute in one frame to an attribute in another frame.

Knowledge Base

That part of a knowledge-based system in which all the domain knowledge is stored, in the form of facts and heuristics.

Knowledge Engineer

A person who works with the expert(s) in order to provide the transition from human knowledge to the appropriate computer representation. Knowledge engineers provide the artificial intelligence (AI) expertise, and play a major role in designing a system.

Rules

Rules consist of a set of conditions, and a set of actions, generally in the form of IF...THEN...ELSE. The conditions are contained in the IF part of the rule, and the actions in the THEN part.

Semantic Networks

A knowledge representation scheme in which information is represented as a set of nodes connected to each other by arcs which represent relations between the nodes.